UNDERGROUND NUCLEAR POWER PLANT SITING

EQL Report No. 6

by

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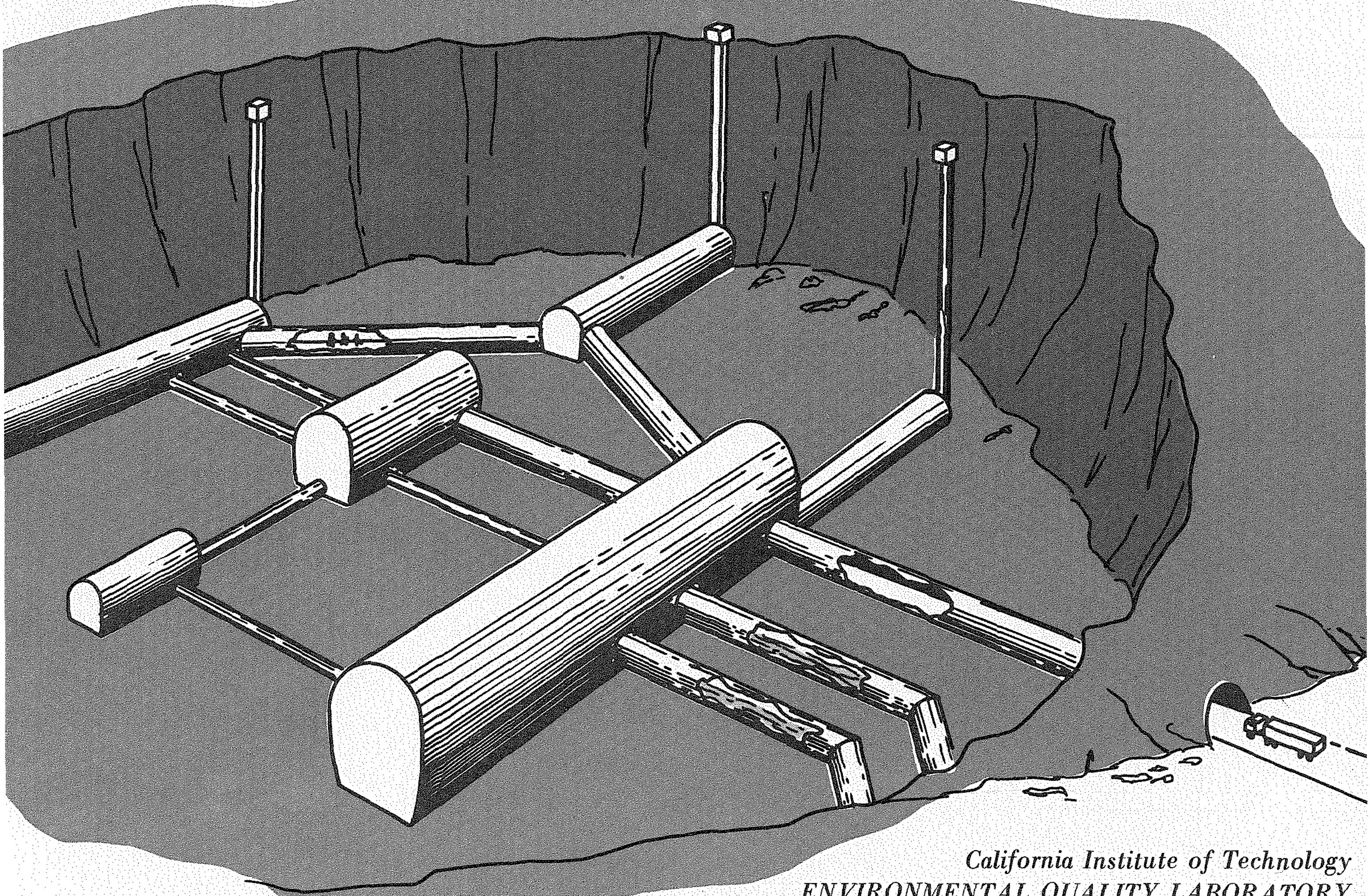
Jointly sponsored by the Aerospace Corporation and the Environmental Quality Laboratory, California Institute of Technology, Pasadena, California 91109



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FOREWORD

This report documents the results of work on underground nuclear power plant siting performed between 23 September 1971 and 28 March 1972 by the San Bernardino Operations of The Aerospace Corporation. This nine-month effort was jointly sponsored by The Aerospace Corporation and the Environmental Quality Laboratory (EQL) of the California Institute of Technology. The EQL support has been provided through California Institute of Technology Purchase Order No. 28-80030-B from Grant No. GI-29726 from the National Science Foundation, Research Applied to National Needs (RANN). These joint studies are continuing, including the possibility of undergrounding the reactor alone, the safety and containment possibilities of undergrounding, and the seismic implications.

This work is part of a continuing investigation by the EQL Task Force on Novel Methods of Siting Nuclear Power Plants, including underground siting, off-shore floating plants, and inland siting. This study of new technological alternatives is itself part of a broader investigation of methods of providing society with a much wider range of alternatives than it now has to cope with the energy demand-supply-environment dilemma.

Lester Lees

Lester Lees
Director, Environmental
Quality Laboratory

August 3, 1972

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SECTION 1

INTRODUCTION

This study is part of a larger evaluation of the problems associated with siting nuclear power plants in the next few decades. This evaluation is being undertaken by the Environmental Quality Laboratory of the California Institute of Technology in conjunction with The Aerospace Corporation and several other organizations. Current efforts are directed toward novel approaches to siting plants within the State of California. This report contains the results of efforts performed by The Aerospace Corporation to provide input information to the larger evaluation relative to underground siting of large central station nuclear power plants.

Projections of electric power demand in California and the country as a whole suggest that a major increase in generating capacity will be required. The problem is complicated beyond that of a large but straightforward extension of capital investment by increased emphasis on environmental factors combined with the early stage of commercial application and regulation of nuclear power sources. Hydroelectric power generation is limited by the availability of suitable sites, and fossil fueled plants are constrained by the availability of high quality fuels and the adverse environmental and/or economic impact from the use of more plentiful fuels. A substantial increase in the number of nuclear power plants is now under way. This source of power is expected to provide the major portion of increased capacity. Other power sources such as geothermal and nuclear fusion are unlikely to satisfy the national needs due to technical problems and the lack of a comprehensive development program.

There are several problems associated with meeting the projected power demand. Chief among these is the location of acceptable and economic plant sites. Indeed a sufficient number of sites may not be found unless changes occur in the procedures for selecting sites, the criteria for accepting sites, or the type of site required. Placement of a nuclear plant underground has been suggested as an alternative to present siting practices. It is postulated that the advantages of underground siting in some situations may more than compensate for added costs so that such facilities could be preferred even where surface sites are available. By virtue of greater safety, reduced surface area requirements, and improved aesthetics, underground sites might also be found where acceptable surface sites are not available.

Four small European reactors have been constructed partially underground but plans for large size commercial plants have not progressed. Consequently, the features of underground power plant siting are not well understood. Gross physical features such as depth of burial, number and size of excavated galleries, equipment layout, and access or exit shafts/tunnels must be specified. Structural design features of the gallery liners, containment structure, foundations, and gallery interconnections must also be identified. Identification of the nuclear, electrical, and support equipment appropriate to underground operation is needed. Operational features must be defined for normal operations, refueling, and construction. Several magazine articles have been published addressing underground concepts, but adequate engineering data is not available to support an evaluation of the underground concept.

There also remain several unresolved questions relative to the advantages of underground siting as well as the costs and other possible penalties associated with this novel approach to siting. These include the degree of increased safety through improved containment; the extent and value of isolation from falling objects, e.g. aircraft; the value of isolation from surface storms and tidal waves; the value of protection from vandalism or sabotage; the extent by which siting constraints are relieved through reduced population-distance requirements or aggravated by underground construction requirements; and the value to be placed upon the aesthetic differences of a less visible facility.

The study described in this report has been directed toward some of these questions and uncertainties. Within the study an effort has been made to identify viable configurations and structural liners for typical light water reactor nuclear power plants. Three configurations are summarized in Section 3. A discussion of the underground gallery liner design and associated structural analyses is presented in Section 4. Also addressed in the study and discussed in Section 5 are some aspects of containment for underground plants. There it is suggested that the need for large separations between the plant and population centers may be significantly reduced, or perhaps eliminated.

Section 6 contains a brief discussion of operationsl considerations for underground plants. The costs associated with excavation and lining of the underground galleries have been estimated in Section 7. These estimates include an assessment of variations implied by different seismic loading assumptions and differences in geologic media. It is shown that these costs are a small percentage of the total cost of comparable surface plants. Finally, the parameters characterizing an acceptable underground site are discussed in Section 8. Material is also included in the appendices pertaining to foreign underground plants, span limits of underground excavations, potential siting areas for underground plants in the State of California, pertinent data from the Underground Nuclear Test Program, and other supporting technical discussions.

SECTION 2

SUMMARY

2.1. STUDY GUIDELINES AND ASSUMPTIONS

Most nuclear power plants now under construction or in the planning stage are of the light water type, either boiling water or pressurized water. Accordingly, the scope of the present study was limited to these types although underground siting may be easily adapted to other reactor types.* The size of the plant to be placed underground was selected to be approximately 1000 Mwe. This size is representative of large units currently available from each of the major manufacturers. It was further stipulated that the packaging and configuration of the underground plant should not require a major redesign of components in the nuclear steam system. This guideline was adopted in an effort to avoid the necessity of a major research and development program on new components before underground siting might be considered.

It was also assumed that the entire plant should be underground. There are, of course, other configurations in which the turbine generator might be left at the surface or in a cut-and-cover pit. Most of the underground European plants are of the type where the turbine generator is at the surface. The limitation of complete burial adopted in this study was judgmental and does not represent the result of analyses or economic trade studies.

Another major assumption in the study was that the underground plant would be in close proximity to the ocean or some other large body of water and utilize once-through cooling.

*For example, the high temperature gas cooled reactor (HTGR) may lend itself easily to underground siting.

2.2 SUMMARY

The feasibility of constructing large underground nuclear power plants is indicated by the partial burial of small European plants and the large underground excavations for hydroelectric facilities. The results of this study confirm this feasibility. It is also found that substantial improvement in containment is a reasonable expectation at many sites. The separation distance from the nuclear plant to population centers might well be reduced from the 10-20 miles characteristic of comparable surface plants to a small localized area. The greater safety implied by this containment might be utilized to permit siting closer to load centers or to locate sites where alternate surface sites are unavailable. The cost penalty associated with underground siting is estimated to be less than 10% and perhaps less than 5% of the total plant cost.

2.2.1 Configurations and Equipment Packaging

The equipment associated with typical surface nuclear plants has been conceptually placed in four principal underground galleries for both pressurized water reactor (PWR) and boiling water reactor (BWR) concepts. Major redesign of the equipment is not required. The four galleries include a reactor gallery (containing the nuclear steam system), a turbine-generator gallery, a nuclear auxiliary gallery (fuel storage, rad waste storage, and processing), and a miscellaneous gallery (control room, stand-by Diesels, switchgear). Smaller openings are provided as appropriate for transformers and control rod drive mechanisms. The spans identified for the reactor and turbine-generator galleries, approximately 100 feet, are large. These span requirements are derived from the assumption that the present surface plant equipment will be used without substantial new research and development. The Churchill Falls Underground hydroelectric power house in Canada, for example, has a span of about 81 feet (see Appendix II). Although there are many natural openings of much larger spans, established excavation practice is limited to the smaller dimensions. The need for large span underground openings implies that sites where good to excellent rock quality is found will be strongly preferred. Excavation in lesser media is judged feasible but at some cost disadvantage.

Three different underground plants have been examined. Two of these represent the straightforward adaptation of surface PWR and BWR plants to the underground site. The third consists of a reconfigured BWR plant in which the pressure suppression emergency system typical of most surface plants has been eliminated. Elimination of this system significantly reduces the volume of excavation and associated costs by utilizing the inherent strength of the rock for containment. Further examination of individual safety systems may lead to further savings by greater utilization of the underground environment.

2.2.2 Liners and Depth of Burial

The large size of the underground openings and concern for safety under earthquake loads implies a need for structural liners. For flat wall galleries these liners can become quite thick. The thicknesses of conventional reinforced concrete liners for horseshoe-shaped gallery cross sections are calculated to be only a few feet. Exact dimensions are specified in Section 4 and vary with the gallery dimensions and assumed seismic loading. Packaging of the equipment to take advantage of the volume provided within the chord of the curved wall was not considered in this effort. If this were done a significant reduction in excavated volume could be achieved.

The depth of burial for an underground plant will depend strongly upon the properties at the specific site. A minimum depth of 150 to 200 feet is indicated for an unlined cavity. This depth should preclude the possibility of opening a crack from the pressurization of the cavity following an accident. For most rock media this depth of burial should also be consistent with nearly complete containment of radioactive leakage within the rock near the cavity.

2.2.3 Containment

The containment of radioactive materials following an accident has been examined for unlined cavities. The reactor galleries included in the three underground plant configurations studied were provided with a liner as part of the rock support structure. This liner could provide a low leakage rate similar to surface PWR plants. Calculations based upon the unlined cavity are, therefore, very conservative.

For many typical rock media the volume and internal surface area of the pore space within a few meters of an unlined reactor gallery will greatly exceed the volume and surface area of the gallery. Steam, from a ruptured reactor coolant line, leaking into these pores will be condensed because of the high heat capacity of the rock. Radioactive particulates and halogens should be adsorbed to the pore surface leaving only the noble gasses to diffuse to large distances. Condensation of the steam will also result in a drop in pressure and eliminate the driving force pushing the gasses through the rock.

If emergency cooling systems similar to those used in surface plants are used underground, pressure in the containment volume should also drop to near atmospheric levels after a few tens of minutes following an accident. Seepage of radioactive products into the rock would essentially stop after this time for lack of a driving force. Simple analyses for dry rock conditions indicate that the distance to which products might have leaked in this time is small for combinations of permeability and porosity of typical rock media. Rocks with large porosity and low permeability are particularly effective. The low permeability restricts the flow and the large porosity provides a large storage capacity. These analyses further imply that little or no release would occur through the rock at the surface following an accident. Presuming adequate air locks can be provided to prohibit leakage through entrance and exit shafts or tunnels, the separation distance between an underground plant and a large population center can be quite small.

2.2.4 Costs

A rough estimate of the cost penalty associated with underground siting has been made based upon 1970-1972 construction costs. The principal cost factors are excavation and liners. Figure 2-1 summarizes these costs for a site where a good quality rock is found and a peak seismic acceleration of 0.5 g is used for the design basis earthquake. Current costs for large nuclear power plants are between $200 and $400 per Kilowatt. The construction costs listed in Figure 2-1 are 6 to 9% of the lower figure and half this much for the upper figure. Other costs will also be incurred for underground plants. These other costs which include such items as operation and maintenance (O & M) are impossible to generalize but are felt to be small. Also, there are cost items that will be credits for an underground plant. For example, cost of the containment structure for a surface PWR might be nearly $2 million using the same unit costs as were assumed for the underground plants. Since the reactor gallery liner is also the underground containment structure an additional containment structure is not required.

Less direct cost advantages might also result by virtue of the improved underground containment and closer siting to load centers. This would reduce transmission line costs and the cost of capacity required to compensate for transmission line losses. These costs are highly dependent upon the specific site selected. The cost of weather protection for outdoor turbine generators used for surface PWR plants would also be avoided. Perhaps the most significant cost saving would be that associated with the reduced impact of weather during the construction of the plant.

2.3 RECOMMENDATIONS

The following recommendations are made as a result of the effort completed in this study. These recommendations are limited to those issues judged to be most important to a preliminary evaluation of the underground concept and do not constitute a comprehensive list of all efforts needed prior to actual construction of an underground plant.

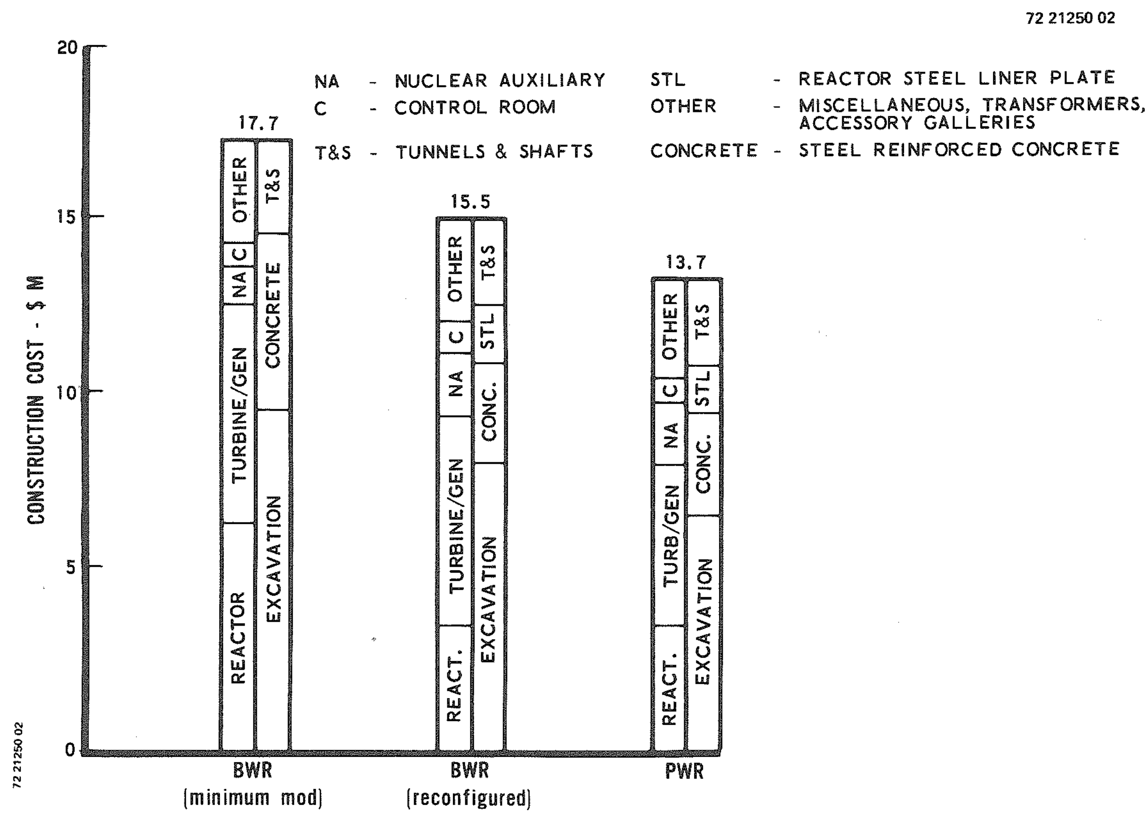


Figure 2-1. Construction Costs

2.3.1 Priority Issues Not Included in Present Study

a. Seismic Protection

One of the key issues in nuclear power plant siting in California today is the approach to aseismic design. This topic was not considered in depth as part of the present study due to limitations of resources and is recommended as part of future investigations.

b. HTGR Configurations

Consideration of power plants using a high temperature gas cooled reactor, HTGR, nuclear steam system was omitted as part of the ground rules of the present study. This reactor concept may have inherent characteristics of improved aseismic design potential in underground plants. The configuration and costing studies of the present effort should be expanded to include the HTGR concept.

c. Underground Sites

The characteristics of the underground sites postulated in this study are summarized in Section 8. If the underground concept is to be pursued, the availability of such sites must be known. A map and literature office study complemented by preliminary field reconnaissance is recommended. The objective of the study would be to locate candidate areas for further study and possible exploration rather than pinpoint exact locations for specific sites.

2.3.2 Further Investigation of Issues Addressed In the Present Study

a. Containment Analyses

The containment analyses conducted in the present study are highly idealized. These should be expanded to include the following:

1) The effect of the gallery liner was not considered. Together, the liner and rock constitute a form of double containment. Furthermore, extensive analyses of more complex containment models are recommended.

2) The steel liner leakage specifications for the reactor gallery may be less demanding as a result of the rock overburden. Relaxed leakage specifications for the steel liner should be examined to determine whether meaningful cost savings might result.

3) The containment analyses should also be expanded to include more detailed calculations of the thermal balance between the escaping products and the rock, the effect of steam condensation, and the degree of entrapment of radioactive products in the rock medium.

4) The potential effectiveness, costs, and procedures for tailoring the media properties at a selected site through grouting, addition of layers, or other means should be examined.

5) Practical engineering of an underground containment system will involve the design and construction of leak-tight penetrations and traps. The design requirements for leakage will be influenced by the number of penetrations and the total plant configuration. Leakage through penetrations was not examined in this study. It is recommended that the effect of these penetrations be evaluated and included in the containment analysis model. Appropriate design measures and analyses to reduce leakage through penetrations should be accomplished in future efforts.

b. <u>Packaging of Plant Equipment</u>

The packaging of the plant equipment to minimize excavation and liner costs consistent with maintenance access was terminated in the present study without iteration. A significant span reduction may also be possible. Further repackaging studies are recommended.

c. Installation Sequence and Access Sizing

The installation of large equipment items was not investigated in detail although no major problem is anticipated. A more detailed examination of this issue is recommended to confirm or deny this judgment.

d. Thermal Stress in Liners

The stress induced in the reactor gallery liner following a maximum credible accident and during start-up operations should be more completely evaluated.

2.3.3 Other Issues Relating to Underground Construction

a. Span Limits

The spans identified for the underground galleries of this study exceed those of similar man-made excavations. These large spans are thought to impact the constructability of the excavation primarily. Further investigation of the engineering limitations imposed by large spans and the corresponding site selection criteria should be conducted.

b. Liner Loads and Response to Seismic Conditions

The loads to be carried by the gallery liners are dependent on the rock quality, seismic conditions and gallery configuration. The present study adopted an extremely simple static loads model. A refined model should be developed that more accurately reflects rock quality and includes dynamic loads imposed by typical seismic conditions. This fundamental rock mechanics issue should eventually include testing and will provide broad benefits beyond underground nuclear power plants.

c. Power Distribution From Underground Sites

The problem of distributing the electrical energy generated in an underground plant was considered only superficially in the present study. This engineering problem should be investigated further to assure no major complications have been overlooked.

2.3.4 Studies Outside the Scope of the Present Effort

a. Alternate Configurations

Judgment was used to establish the burial guidelines of the present study. These guidelines exclude plant configurations in which some of the equipment is left at the surface or in cut-and-cover excavations. These alternatives should be investigated.

b. Advanced Reactor Concepts

The advantages of underground siting may be more significant or attractive for advanced reactor concepts such as breeders. The siting issues and potential for underground siting of such plants should be examined.

c. Inland Underground Siting

The present study considered only those sites where once-through cooling could be accomplished. The siting of underground plants at inland sites where other forms of cooling are required should also be investigated.

d. Specific Site Point Design

The preliminary examination of selected issues in the present study leaves many questions unanswered. At some point in the future it may be desirable to undertake a comprehensive preliminary design of a specific plant type at a selected site. Such an effort would greatly enhance understanding of the technical and engineering issues of underground power plant siting and provide a more substantive basis for projecting costs and schedules.

e. Multiple Plant Sites

The siting of several plants and supporting facilities at a single location should be investigated. This might include the power park concept in which fundamental questions relating to the functions (e.g., degree of fuel processing) to be included at the park should be examined.

f. Underground Pumped Storage Plants

The development of underground sites to include pumped storage facilities has been proposed. This concept may have merit and is recommended for further evaluation.

SECTION 3
UNDERGROUND CONFIGURATION AND LAYOUTS

The feasibility of constructing large underground nuclear power plants is not seriously questioned. In fact, several small plants have been built partially underground (see Appendix 1). However, the practicality and desirability of constructing large plants is not established. An evaluation of the practicality or desirability of such plants must be based upon a reasonable understanding of how they might be designed and configured. This section summarizes three configurations developed to be consistent with the rather stringent study guidelines described in Section 2. 2. The three configurations include straight-forward adaptations of a surface pressurized water reactor system, a boiling water reactor system, and a reconfigured boiling water reactor system in which the pressure suppression containment system typical of surface BWR plants has been eliminated. Many other configurations are equally feasible and one or more of these might well prove to be preferred. The three configurations discussed here are not optimized but are thought to provide a basis for estimating representative costs. They are the product of only limited iterations between an initial concept and an evaluation of the design consequence. It is felt that these three plant configurations provide a basis from which other configurations might be identified and from which broad conclusions might be derived pertinent to technical issues, site criteria, cost penalties, operational problems, and potential safety features.

3.1 APPROACH

Significant differences exist in the physical dimensions and geometries of the principal steam and power generation components of lightwater reactor systems as supplied by the major manufacturers. These properties significantly impact the design of an underground plant. In order to specifically identify the differences, the design documentation for several nuclear plants using boiling water or pressurized water nuclear steam systems was reviewed.

As a ground rule it was decided that the "as is" manufactured shape and dimensions of the nuclear steam system (NSS) and turbine generator (TG) be retained in the underground plant design thereby insuring that the operation and performance of the plant would closely approximate that of a surface plant. Also, a design goal was adopted that the clear span of the underground chambers be limited to less than 100 feet and preferably to 60 feet for the most critical chambers. Both of these dimensions represent judgments influenced by current underground construction practice and are not the result of some physical or engineering limit. The limited span goal together with the as-is equipment ground rule caused some difficulty in developing reactor chamber designs. This difficulty and its resolution are discussed in a subsequent section.

The most apparent advantage for underground power plant siting is greater safety through improved containment. This objective can probably be met without the need to bury the entire plant. As is the case for several European plants of much smaller size than considered in this study, the reactor and other nuclear systems might be placed underground with the turbine generator, control room, and miscellaneous support facilities at the surface. However, additional benefits may be obtained in addition to improved containment if the entire plant is underground. Objectionable use of ocean front land for the surface facilities would be eliminated. A plant could be placed further inland with lesser likelihood of aesthetic land use objections at the expense of longer cooling water tunnels. In most locales an inland site with a surface turbine would require several hundreds of feet separation in elevation between the reactor and the turbine or a need to pump cooling water through a similar head. The large reactor-turbine separation would result in some loss in plant efficiency due to the increased length of the steam lines. A surface turbine generator or other surface facilities would also be exposed to the elements including tornadoes and tsunamiis and potential accidents such as air crashes. For these reasons the entire plant was placed underground for all three configurations defined. Costs for some combined surface and subsurface plant configurations may be estimated by noting the costs associated with each underground gallery, Figure 2-1. A further examination of such plants was not included in the present study.

Many problems with varying degrees of difficulty are encountered in the formulation of a plant configuration with trade-off of many issues required. Not all of the work so defined could be treated to the depth desired. Therefore, it was necessary to treat the various issues on a priority basis as time and resources permitted and to select approaches to some issues only on the basis of judgment. Some of the issues raised during the study but not treated in depth were:

a. The Number of Underground Galleries

It is conceivable that the entire plant could be placed in a single large gallery or in several smaller galleries. The large gallery would probably be very long since large spans would impose unreasonable siting requirements or construction problems. The need for personnel access to the turbine generator, auxiliary equipment, miscellaneous support equipment, and the control room during normal operations and following an accident dictates against the single gallery concept. The partition of equipment into four major galleries was derived from an examination of surface plants. The potential for venting the reactor to one or more of the other galleries to minimize internal temperatures and pressures was not considered. The capability of the rock medium to contain the pressures and temperatures in the reactor gallery alone appears to be quite adequate and the contamination of larger volumes was, therefore, felt to be unnecessary.

b. The Separation and Orientation of Underground Galleries

The required separation of large underground cavities is highly dependent on the rock medium. Current civil practice typically indicates a separation of perhaps one-fourth to one-half the span of the largest gallery. Military excavations utilize larger separations because of the much larger stress levels anticipated at structures subjected to nuclear attack. The separations adapted in this study are larger than many civil projects but slightly smaller than military considerations dictate. The separations are taken as approximately twice the gallery span for sizing interconnecting tunnel length and are thought to be conservatively large.

The orientation of the major galleries is such that the major axes are parallel. It is assumed that the joint pattern at the site will be such that a preferred orientation is identifiable so that large spans can be utilized.

No major advantage or disadvantage could be identified as to whether all galleries should be at a common elevation or whether a stacked orientation would be preferred. There is a desire to keep the reactor close to the turbine and the turbine at or near sea level to minimize steam losses and cooling water pump requirements. This desire can be satisfied by either a stacked or single elevation layout. The single elevation layout was adopted as more closely resembling a surface plant.

c. Plant Electrical Interfaces

Several electrical interfaces of differing power capacities are required for a large nuclear plant. While the lower capacity interfaces have stringent electrical reliability requirements, for an underground plant the main power output circuit is the more complex and difficult to design. For economic reasons, the low voltage generator output leads are kept as short as practical by transforming the generator voltage directly to the transmission voltage level with a transformer located near the generator. This is easily achieved in the design of surface plants. The reliable transmission of the plant electric power usually necessitates that the plant interface electrically with the local system grid at a substation. For large plants, the electrical interface is complex making the substation large in size; in many instances occupying surface areas larger than the plant buildings. The placement of a large switch yard at grade near the plant is inconsistent with the overall underground plant design objectives. The prospect of using gas insulated bus and switches could reduce the substation size. The placement of all substation equipment underground was also considered but incurs an economic penalty which is dependent upon transmission requirements rather than generation and as such was not studied. The most probable location of the main generator transformer is in a separate underground gallery near the generator which is large enough to accommodate additional electrical equipment for transmission of the circuit to the surface. Several circuit design options are available and no unusual problems are expected.

d. Access Shafts and Ventilation

The plant layouts include provisions for personnel and equipment access into the galleries from the surface. The locations and sizes of gallery entrances were established on the basis of plant equipment dimensions, the locations of the largest single equipment pieces, and the anticipated entrance needs of plant operating personnel. The main personnel access to the control room was thought best to be a shaft with an elevator. This would provide speedy and safe access to the control center area. A tunnel would provide the most convenient entrance for installation of large and heavy plant machinery. A tunnel entrance of 30 foot diameter would permit the handling of the largest pieces with existing transporters and eliminate some of the handling of turbine generator equipment within the gallery if installed through a shaft. The use of a shaft as an equipment entrance would present a significant departure in heavy equipment handling experience. A large capacity crane (200-600 tons) with a long hook reach would have to be specially manufactured and installed for each site. Several other shafts or tunnels are required to provide for air ventilation; emergency personnel escape; electric power, water, waste; fuel handling; and safety valve steam exhaust. The selection of shafts and tunnels or both is obviously dependent upon site location and individual site terrain features. An example of a possible configuration assuming all shafts as a basis for conservative costing is included in the plant layout drawings but is not implied to be optimum for the particular layout.

3.2 PRESSURIZED WATER REACTOR (PWR) PLANT CONFIGURATION

The 1060 Mwe PWR plant under construction by Pacific Gas and Electric Company at Diablo Canyon and the proposed 1100 Mwe unit No. 2 at San Onofre by Southern California Edison were selected as being representative of current design (1969) large size PWR power plants. When viewed externally both plants have similar appearances, e.g., each plant has a reactor building in the shape of a large cylinder with a diameter of about 140 feet and a height of about 200 feet. Connected to the reactor building are two rectangular structures comprising the TG building and nuclear auxiliary buildings arranged as shown in Figure 3-1. The nuclear steam system (NSS) components are principally located within the reactor building and are functionally identical in each plant except in physical geometry. The Diablo Canyon Nuclear Steam System as supplied by Westinghouse is a four (4) primary loop coolant design with four steam generators and pumps. The steam generators, primary coolant pumps, and piping are arranged symmetrically around the reactor pressure vessel as shown in Figure 3-2a. The minimum lateral dimension of this NSS is 65 feet. The span of an underground cavity to contain this system would be in excess of 80 feet. It appears that a modification to the NSS as shown in Figure 3-2b could reduce the minimum diameter to 59 feet. The San Onofre unit 2 NSS as designed by Combustion Engineering (CE) also uses a four-loop primary coolant system as shown in Figure 3-3. However, this system uses only two larger sized steam generators. The maximum diameter of the Combustion Engineering (CE) NSS is about 54 feet. Because of a shorter chamber span requirement, the CE NSS was selected as a baseline design for the underground plant NSS layout. Data on the geometry of the Babcock and Wilcox NSS was examined and found to be similar to the CE NSS system design.

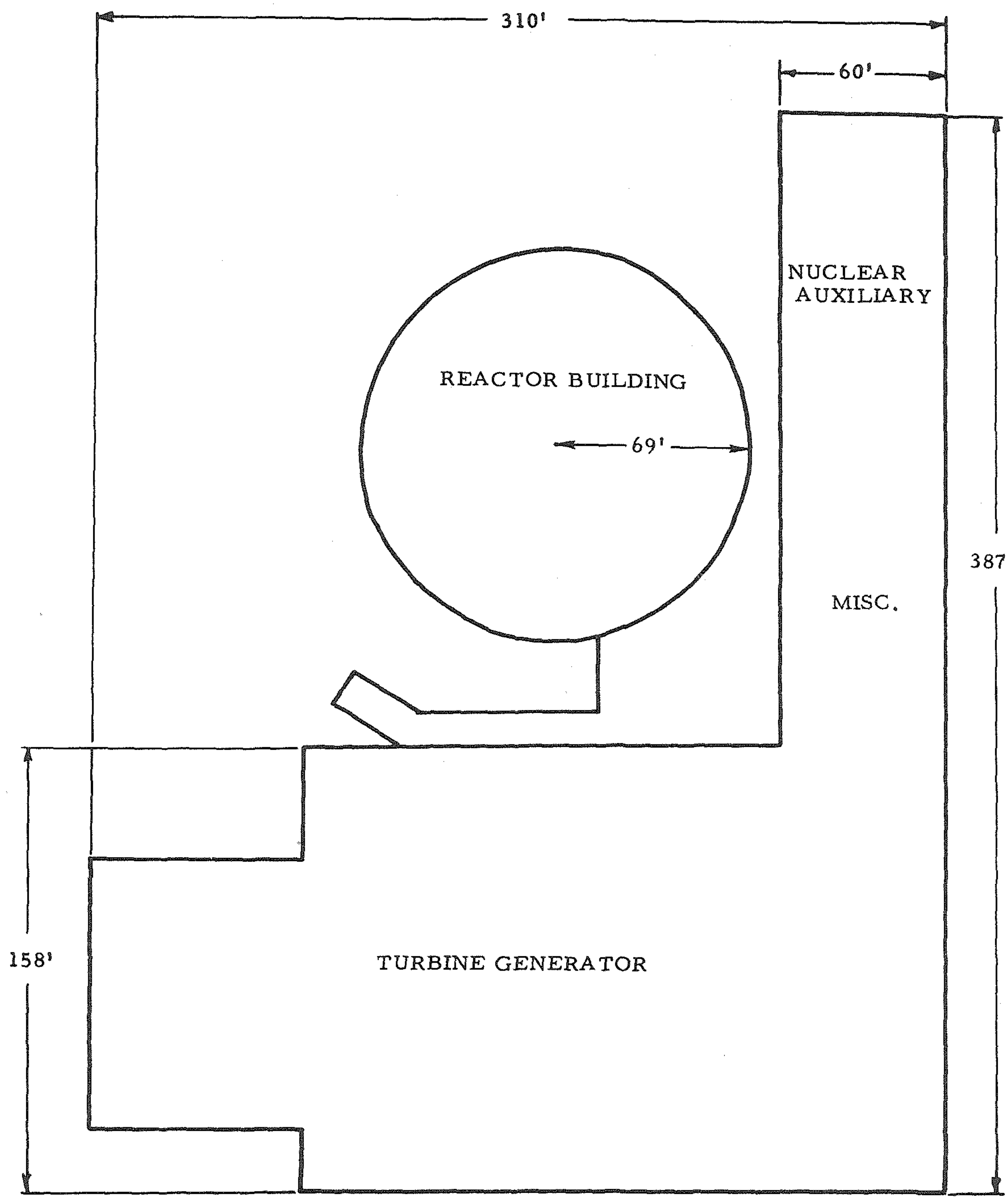


Figure 3-1. PWR Above Ground Plant Layout (Typical)

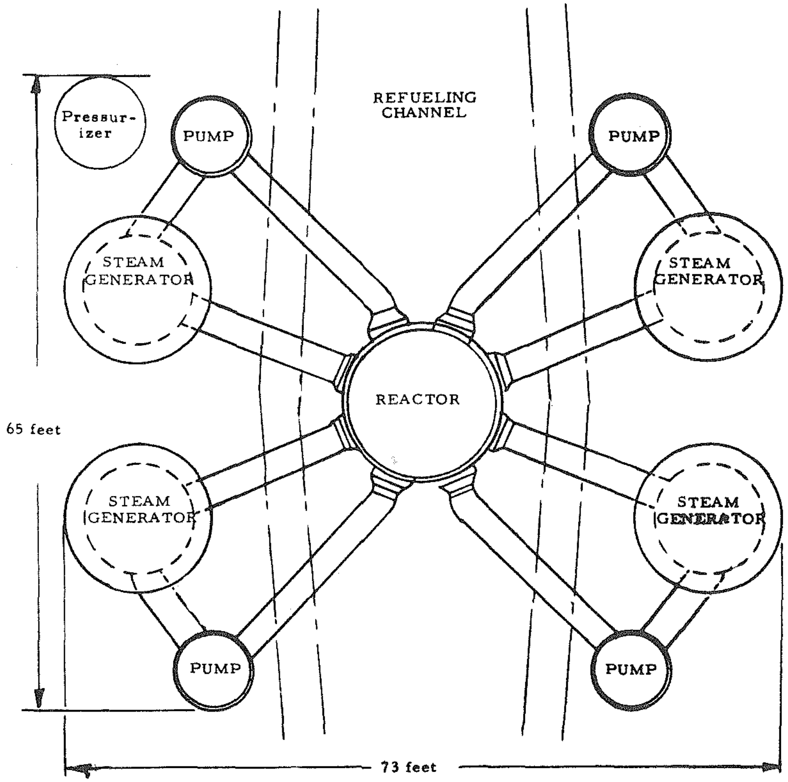


Figure 3-2a. Westinghouse NSS, 3300 Mwt; Plan View

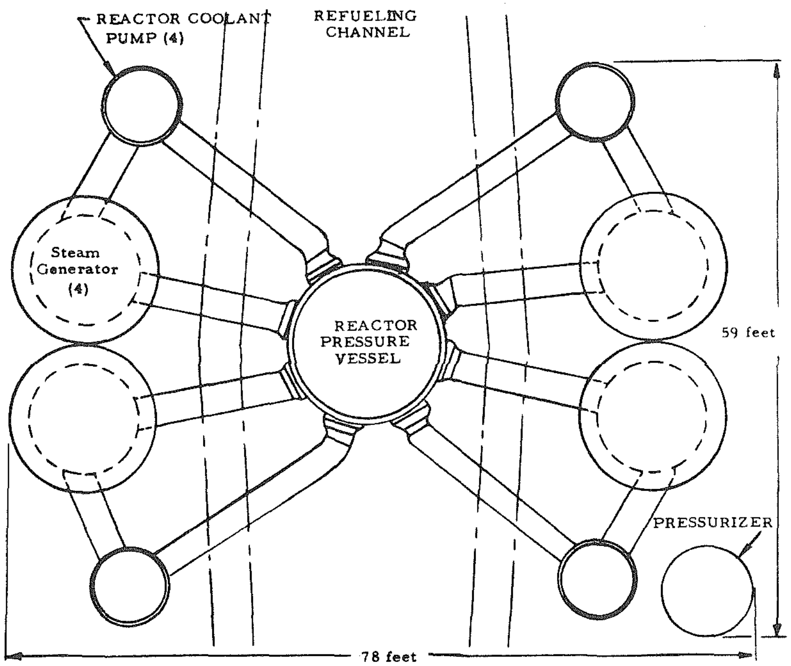


Figure 3-2b. Modified Westinghouse NSS, 3300 Mwt; Plan View

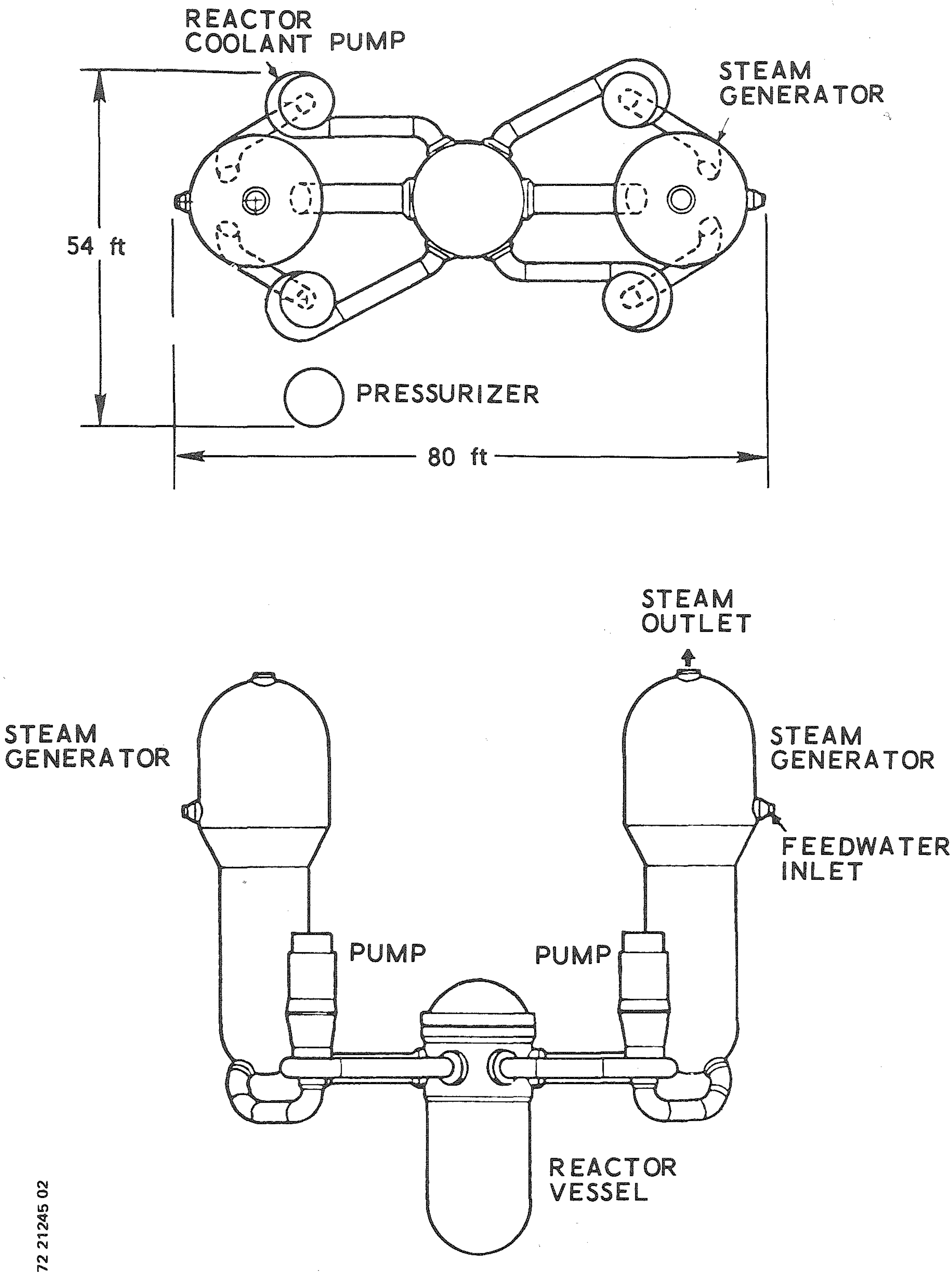


Figure 3-3. Combustion Engineering NSS

After some consideration for the space required for reactor refueling, maintenance, equipment installation, access for replacement, and the incorporation of current engineered safety systems, it was estimated that a 3300 Mwt (1000 Mwe) PWR nuclear steam system could be installed in a chamber with a 60 foot span as shown in Figure 3-4a. The NSS chamber length would be 120 feet. A height of 135 feet would be provided to include clearance for a large bridge crane to handle heavy components. Design of the NSS foundations can be accomplished in several ways as influenced by considerations for seismic motion isolation. One method would be to utilize the rock as a natural foundation material, as shown in Figure 3-4b. The rock chamber is excavated and the equipment grout plate mounted directly to the rock floor. Another method is to excavate the entire chamber and construct a concrete foundation similar to the surface plant.

The emergency core cooling and containment spray pumps and valves located immediately external to the reactor containment structure for surface plants are located in the Nuclear Auxiliary (NA) chamber. This NA chamber is sized to contain all equipment, except liquid rad waste storage tanks, now found in the auxiliary building of surface plants. The stack typically used by surface plants to discharge gaseous waste has been replaced by a rad waste disposal system such as the freon cryogenic system currently being developed by the AEC. To accommodate the additional waste disposal equipment, a 5% increase in size was added to the NA building. A departure from surface plant design is the required increase in length by about 50 to 100 feet of the fuel transfer tube between the reactor and spent fuel pit located in the NA chamber.

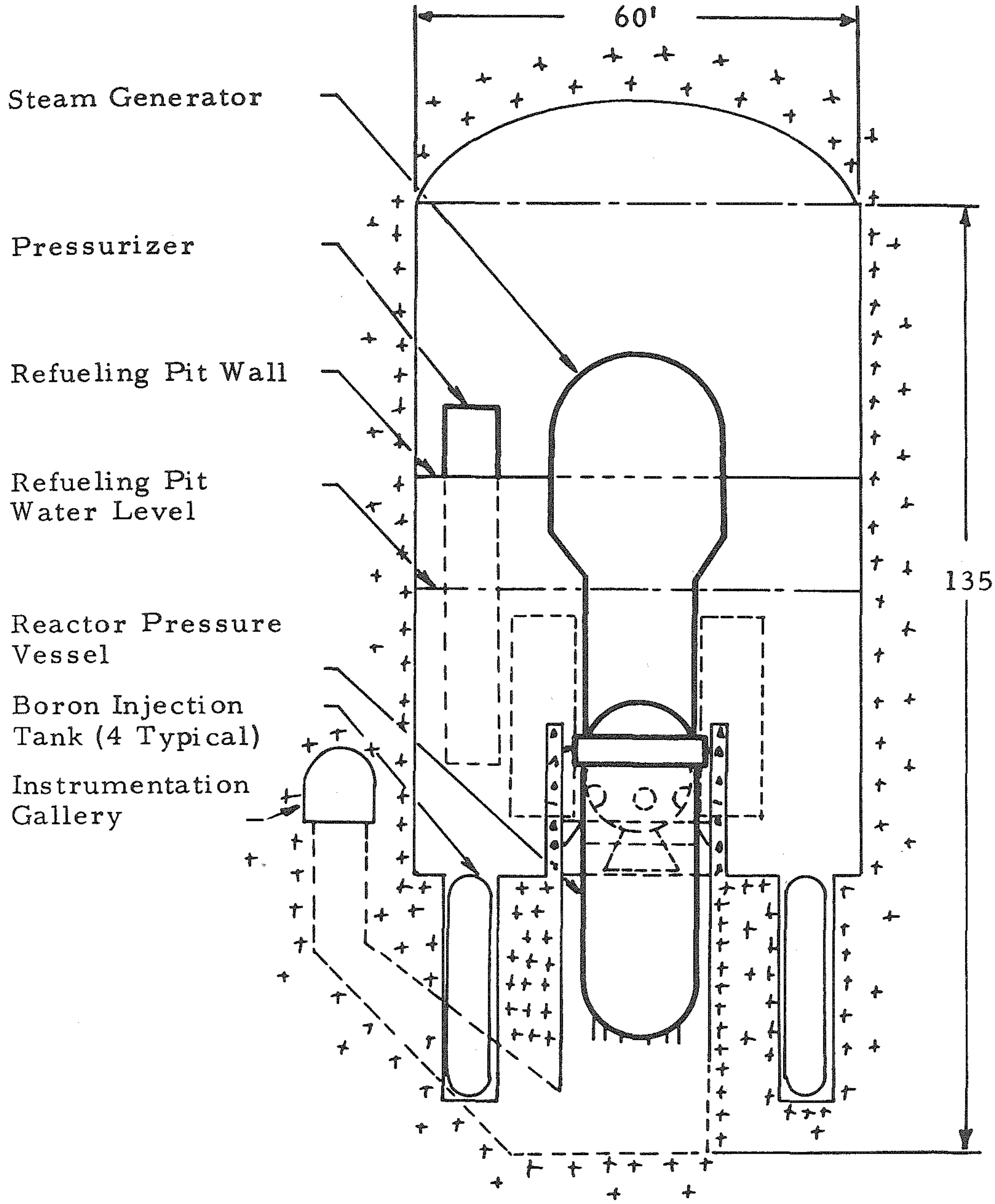


Figure 3-4a. PWR Reactor Chamber (Elevation View)

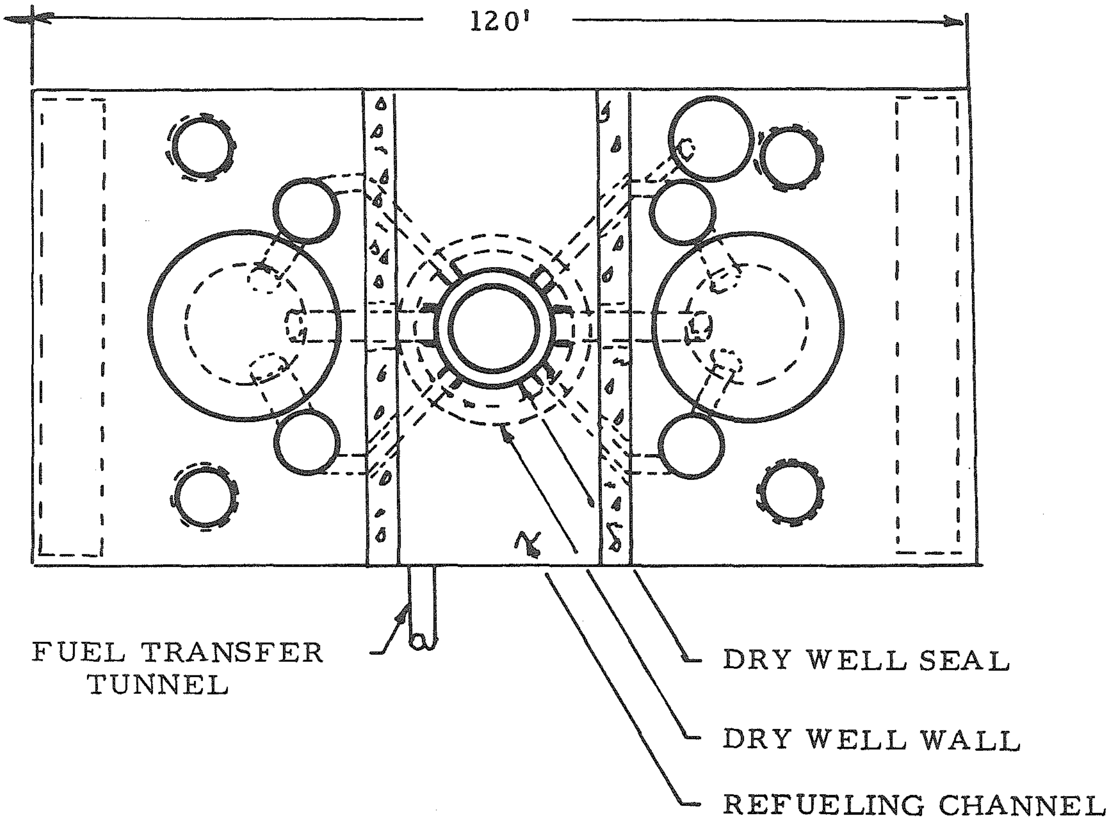


Figure 3-4b. PWR Reactor Chamber (Plan View)

The turbine generator (TG) chamber is the largest single excavation of this plant. The TG chamber dimensions are 355 feet long, 90 feet wide, and 100 feet high. Forty feet of the length is for access and maintenance space. The minimum chamber span of 90 feet is determined by the turbine foundation width and the low pressure stage steam connections. In addition to the turbine and generator, the interstage moisture separator and steam reheater valves and piping are located above the turbine floor next to the turbine. It is expected that this area will be somewhat crowded with all equipment in place and somewhat greater width would be desirable to achieve less dense packaging. Connections to the turbine condenser and other miscellaneous equipment are located below the turbine floor. The location of the several feed water heaters and fresh water condensers has been moved from the TG chamber and located in separate chambers (see Figure 3-5) adjacent to the TG chamber.

The remaining plant equipment including all emergency power sources, switch gear, refueling water, and condensate make-up are contained within the miscellaneous equipment chamber. A sample layout of the plant is shown in Figure 3-9 of Section 3. 5.

3.3 MINIMUM MODIFIED BOILING WATER REACTOR (BWR) CONFIGURATION

The BWR Nuclear Steam System departs significantly in physical size and configuration from the PWR system. Of the two, the BWR NSS is smaller in size and more compact although the reactor pressure vessel itself is larger. The reduction in overall size is achieved through elimination of the steam generator and interconnecting primary loop pumps and piping of the PWR. Principal connections to the BWR pressure vessel include the feed water intake, steam lines, and core water recirculation pump lines. The recirculation pumps are mounted

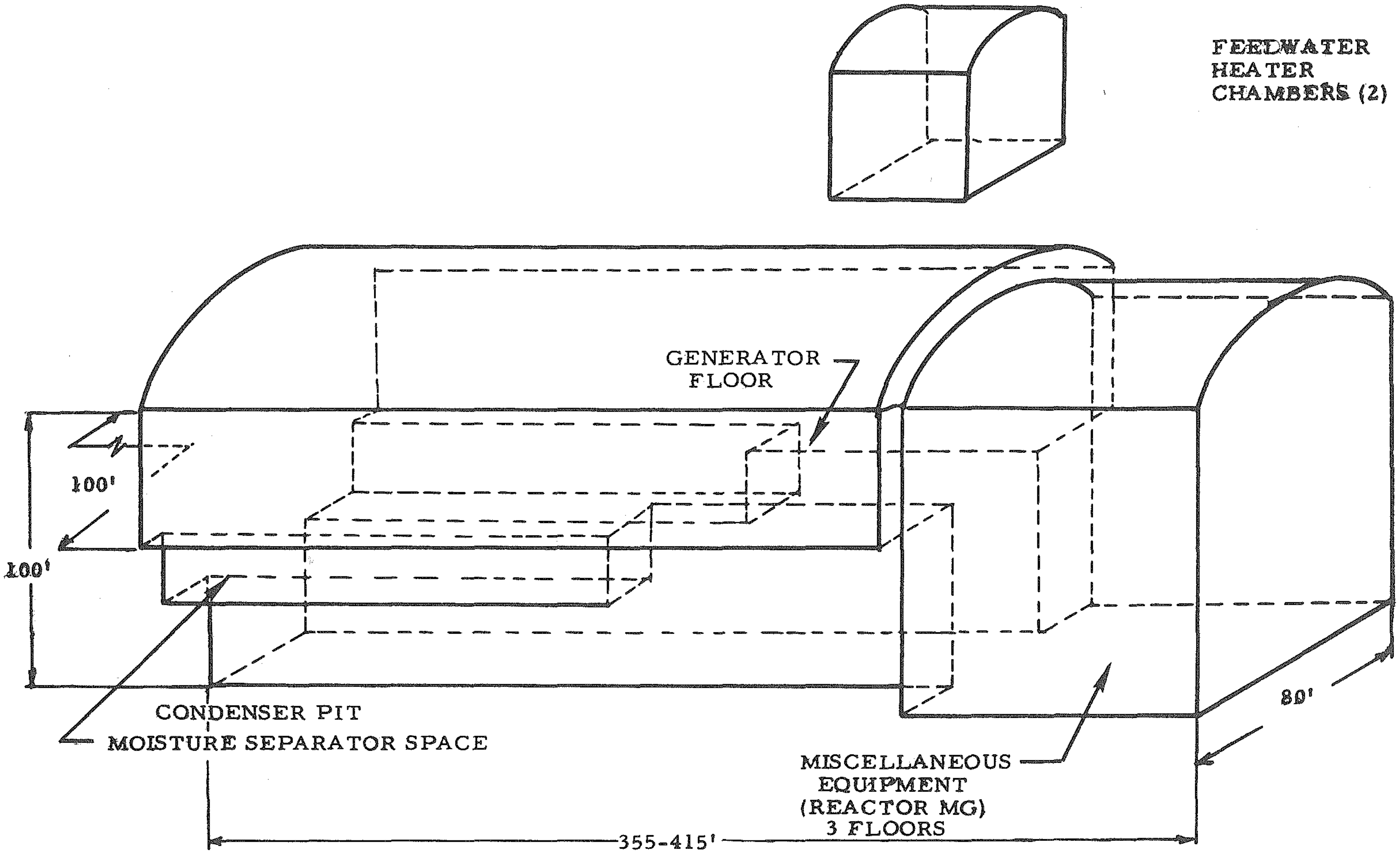


Figure 3-5. Turbine-Generator Chamber (Typical)

along side the pressure vessel at the base. In plan, the reactor pressure vessel is symmetrical and including all attachments has a maximum diameter of about 50 feet in a plane through the level of the recirculation pumps.

Historically, the development of the BWR NSS has included the development of a pressure suppression containment. This containment has been evolving toward greater plant energy density with lower construction costs. Beginning with the Dresden I plant the containment design began as a sphere, then evolved into a cylinder, then to the inverted light bulb shaped drywell and torus and more recently the over/under designs typical of the Limmeric and Zimmer plants. The various phases of containment development are illustrated in Figure 3-6. From an inspection of Figure 3-6 it is apparent that the minimum width of these BWR NSS containment designs exceed the 60 foot chamber span design goal. After rejecting several alternative methods the design of Figure 3-7 with a span of 75 feet was selected. The design uses the inverted light bulb shaped drywell design with dimensions estimated from the Quad Cities and Brown's Ferry plants. The design shown in Figure 3-7 includes a reconfiguration of the toroidal suppression pool storage tanks and drywell interconnecting piping. The equipment within the surface plant reactor building would be retained but reorientated to fit the shape of the underground BWR NSS gallery.

A significant difference between the BWR and PWR chamber designs described above is the elimination of the BWR gallery leak-tight steel liner since only the drywell and suppression tanks are exposed to the accident pressure transient.

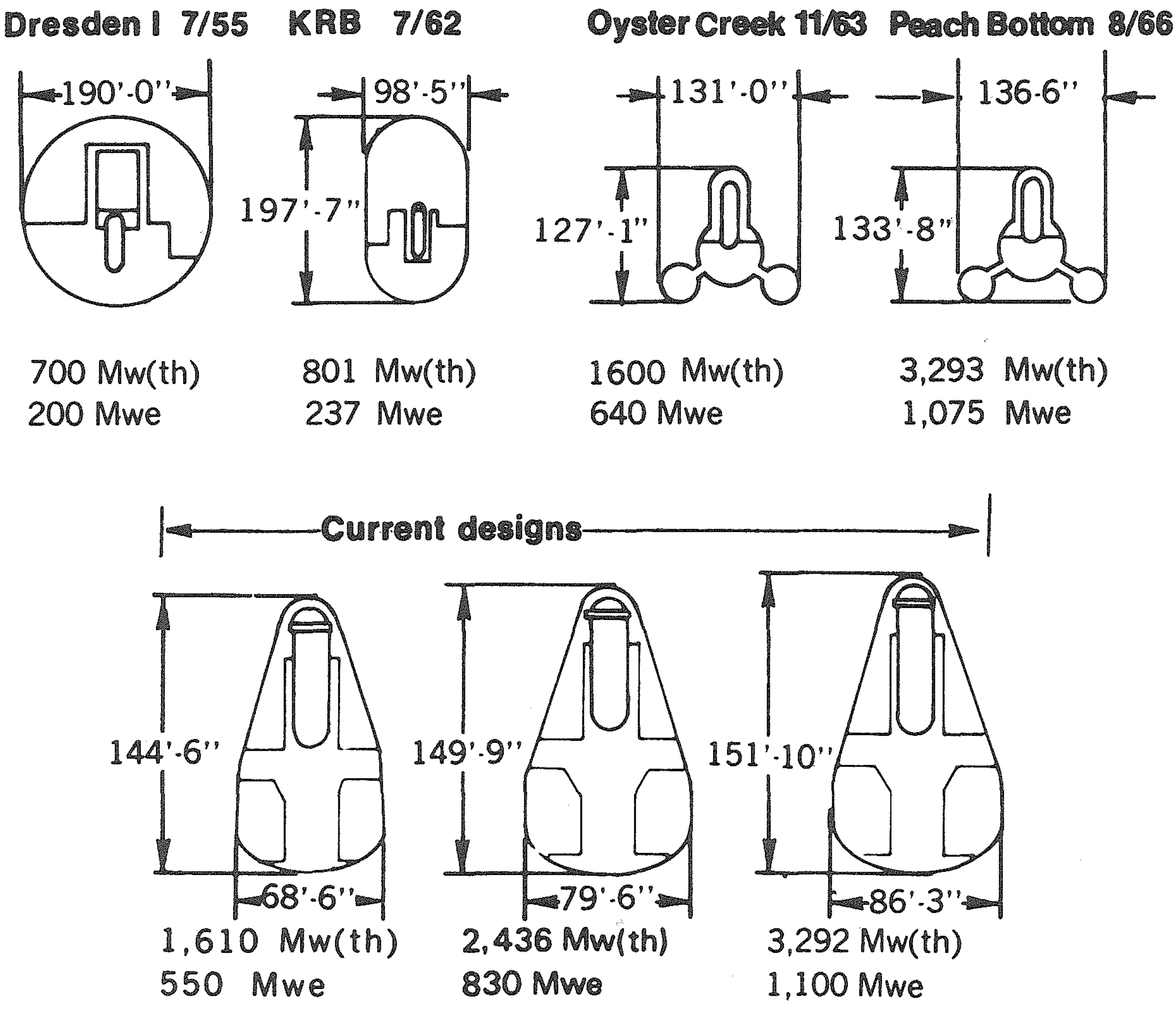
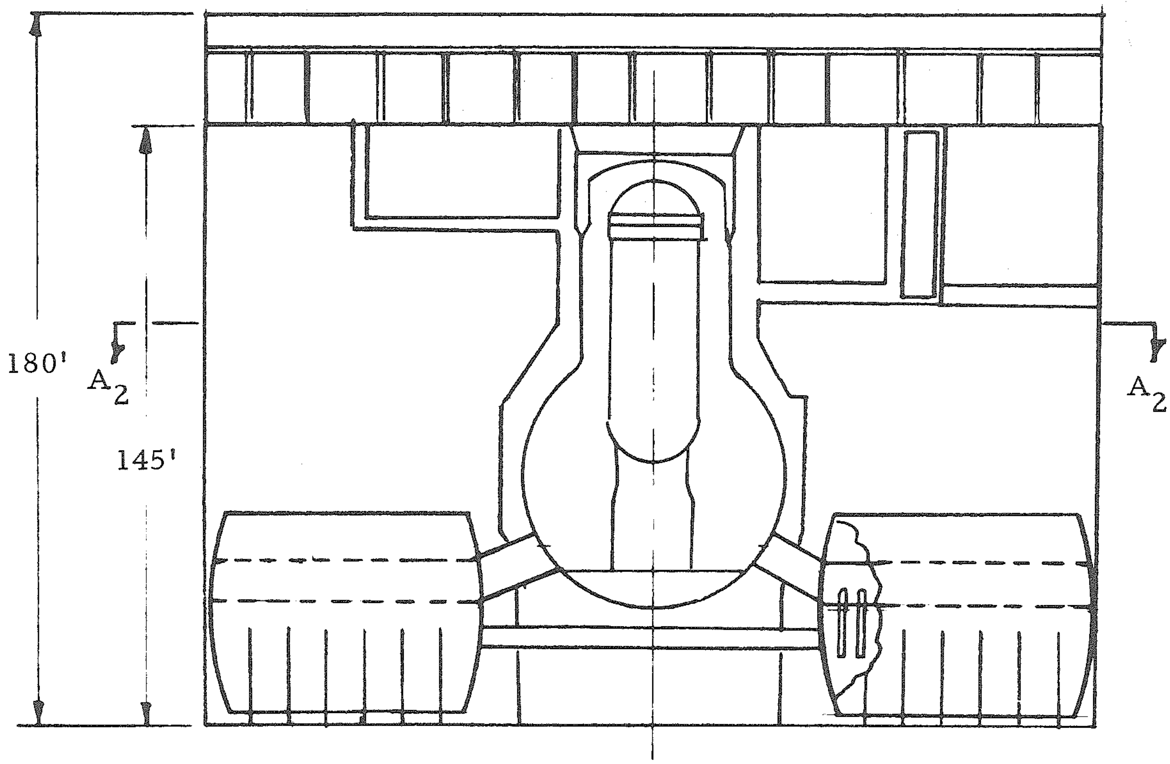


Figure 3-6. Evolution in BWR Plant Design



Section B_2-B_2

Figure 3-7a. Minimum Mod BWR (Elevation View)

Section A_2-A_2

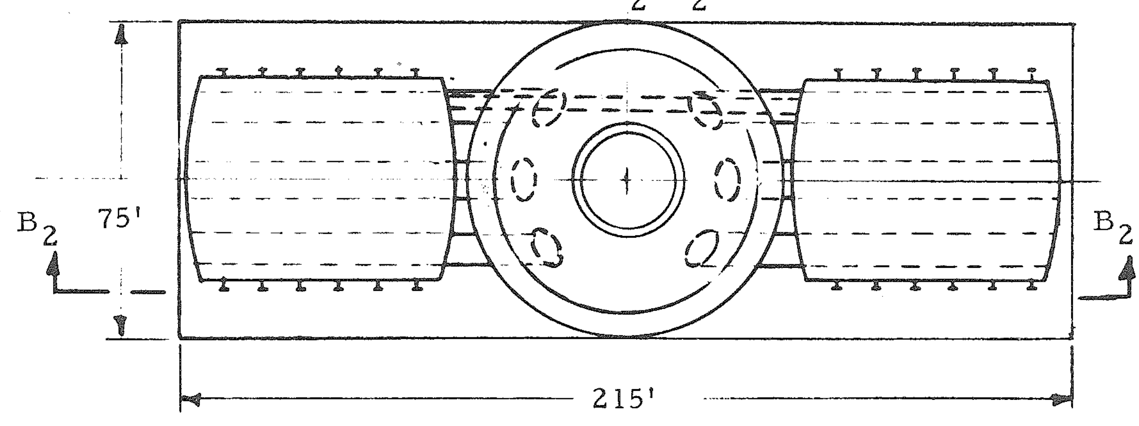
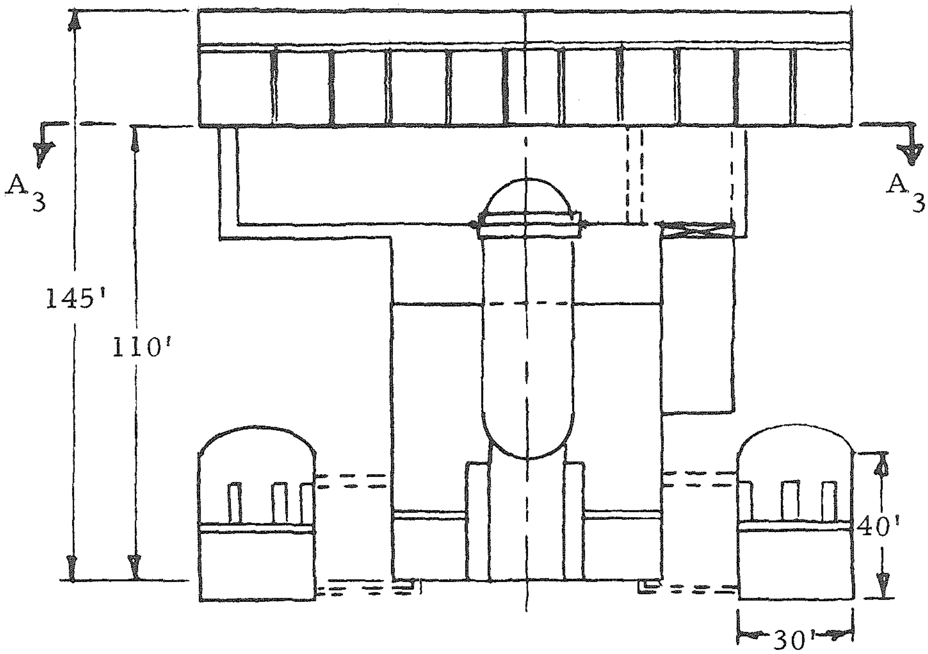


Figure 3-7b. Minimum Mod BWR (Plan View)

The turbine generator, nuclear auxiliaries, and miscellaneous equipment galleries comprise the remainder of the plant. The TG used as a basis for chamber sizing was a General Electric design. Examination of the G. E. design concluded that the turbine could be placed in an 80 foot wide chamber if the turbine radiation shield were eliminated. With shielding, the chamber would be 100 feet wide at the turbine floor. The BWR TG chamber is longer than the PWR TG chamber (415 feet) to accommodate items peculiar to the BWR plant. Feed water heaters and fresh water condensers are located in a smaller chamber adjacent to the TG chamber.

3.4 RECONFIGURED BWR PLANT CONFIGURATION

The reconfigured BWR plant was developed in an effort to make greater use of the surrounding rock consistent with present reactor systems. The layout as shown in Figure 3-8 would place the reactor in a lined vertical cylinder excavated in the rock. The cylinder would be about 65 feet in diameter. An excavated chamber would extend above the reactor high enough to accommodate the handling of equipment and reactor shields with a large crane. The chamber width would extend to the diameter of the reactor excavation and of sufficient length for a refueling channel. Separation of the refueling channel and drywell would be with a seal between the rock excavation and the reactor pressure vessel. Access to the drywell for equipment repair and inspection would be accomplished through a shaft along side the pressure vessel excavation. The construction of this NSS would include a steel liner similar to the PWR plant.



Section B_3-B_3

Figure 3-8a. Reconfigured BWR (Elevation View)

Section A_3-A_3

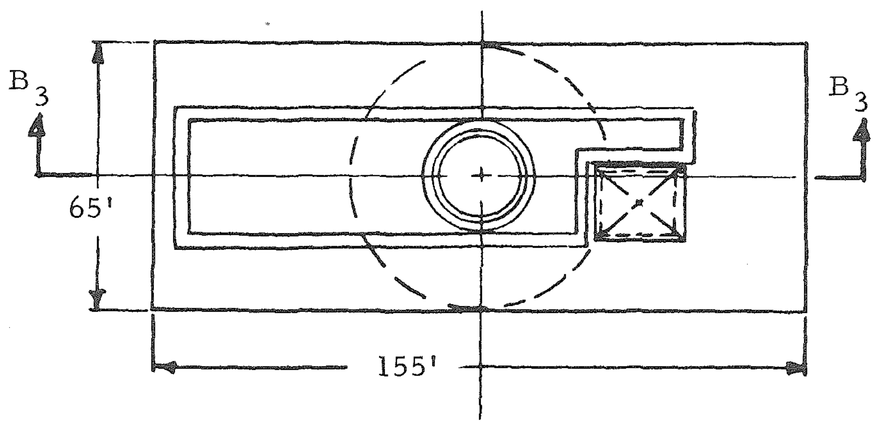


Figure 3-8b. Reconfigured BWR (Plan View)

Separate galleries adjacent to the reactor gallery contain the control rod drive and emergency core cooling pump equipment. Reactor emergency cooling water is supplied from storage tanks located in the nuclear auxiliary gallery. Although the increase in size of the nuclear auxiliary gallery partly offsets the reduction in size of the reactor gallery, the smaller reactor gallery is thought to be a significant improvement over the minimum modified BWR plant configuration.

The remaining portion of the plant would have nearly the same layout and dimensions as the minimum modified BWR described in Section 3.3.

3.5 SUMMARY

The foregoing configuration descriptions present layouts for three underground reactor power plants, two of which are BWR plants and one a PWR plant. Each plant consists of several chambers which contain the various equipment of the plant. The significant features of the plants are that they use current designs for the principal components of the nuclear steam system and turbine generator. Improved radioactive waste collection and retention systems have been assumed to be part of the nuclear auxiliary facilities to eliminate the necessity for a plant stack. Retention of the emergency safety systems, particularly those used to reduce post accident containment chamber pressure and radioactivity implies a very conservative approach to safety since burial may reduce the need for some of these features by virtue of the containment provided by the rock. This subject is discussed in Section 5 of this report. A summary of the dimensional data for each of the three plant chamber layouts is given in Table 3-1.

TABLE 3-1

NUCLEAR PLANT GALLERY MINIMUM REQUIRED EXCAVATION DIMENSIONS

(H' x W' x L')

Chamber	BWR (Minimum Mod)	BWR Reconfigured	PWR
Reactor	180 x 75 x 215	145 x 65 x 155	135 x 60 x 120
Control (2)	- - -	40 x 30 x 50	10 x 10 x 30
Nuclear Aux.	90 x 50 x 155	90 x 50 x 255	90 x 50 x 260
Turbine Generator*	100 x 100 x 415	100 x 100 x 415	100 x 90 x 355
Feed Water Heaters (2)	45 x 40 x 50	45 x 40 x 50	45 x 40 x 50
Miscellaneous (excludes Control Room)	30 x 60 x 120	30 x 60 x 120	30 x 60 x 120

* Includes 40 feet of gallery length for construction and maintenance

The excavation dimensions are gallery sizes defined above and influenced by dimensions of the structural liner. As shown in Section 4 the roof and wall structure may include an arch design and that the height of the arch would be additive to the clear span dimensions of Table 3-1. The additional volume provided by the curved wall and roof liners could be utilized to achieve a further economy in plant packaging. Iteration of the plant packaging to make use of this volume was not possible in this study. Further packaging studies of this type are expected to reduce the cost of underground siting and are recommended as part of future study efforts. As indicated in Section 3.1, no factors were identified to specify the number or orientation of underground galleries. Peculiarities at a particular site will more than likely dominate selection of the layout. Figure 3-9 was constructed to illustrate one possible layout wherein only a single level is used and large intergallery rib spacing is provided. This figure indicates that the total plant would be included in a 700 by 700 foot area of approximately 11 acres exclusive of cooling water conduits and headworks. The geologic medium containing the plant would be slightly larger. Since a conservatively large intergallery rib spacing was assumed it may be possible to reduce these dimensions without the need to use a stacked configuration.

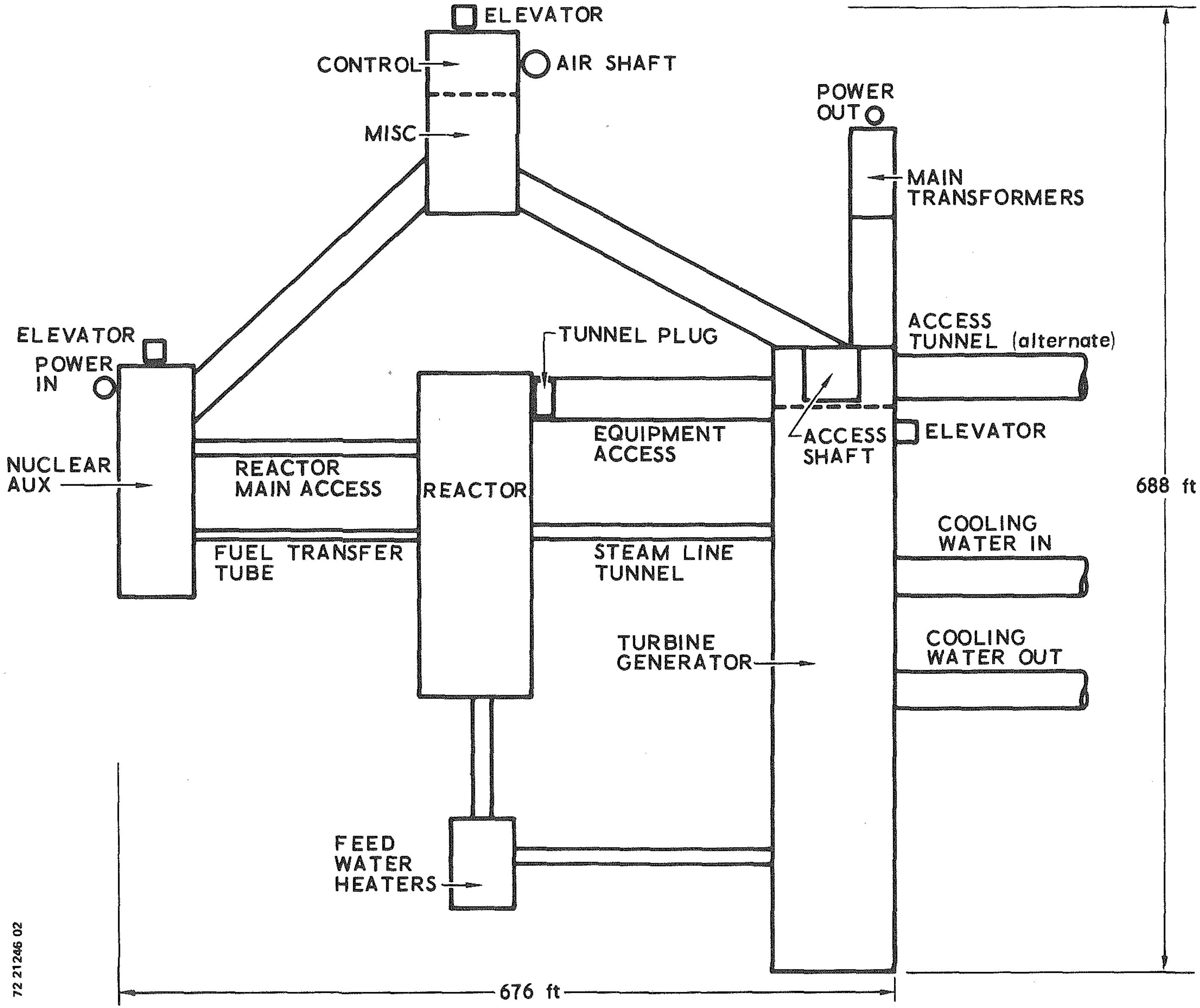


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Figure 3-9. Sample Layout Underground PWR Nuclear Power Plant

SECTION 4

ENGINEERING ANALYSES

Several engineering analyses were performed to further define the plant configurations discussed in Section 3. Early studies were completed to identify the depths of burial that might be dictated by structural considerations. Subsequent studies of the seepage of fluids through the rock indicate that similar depths are probably more than adequate to provide excellent containment. Although the structural analyses were highly idealized, the resulting depths are not large and a reexamination of the analyses was not made to determine a more precise minimum depth.

The bulk of the analyses conducted was directed toward the definition of the gallery liners as dictated by the quality of the rock medium at the site and the design basis seismic loads. This work included a large number of parametric variations and resulted in a decision to adopt a horseshoe cross sectional shape rather than the flat wall and arch typical of most underground hydroelectric excavations.

It was originally planned to address the relative response of underground and surface plants to seismic loading conditions. Unfortunately, the limited resources and time available for the study did not permit these investigations to be completed. This subject continues to be of great interest and is recommended to be included in subsequent studies.

4.1 DEPTH OF BURIAL

One of the potential advantages of placing a nuclear plant underground is the increased containment of radioactive materials afforded by the medium. Aboveground plants are equipped with secondary containments of either concrete or steel which provide a passive barrier to atmospheric dispersal of radioactive material in the event of an accident. For a plant sited below ground, the rock affords an additional barrier. However, for the rock to be effective the depth of burial must be sufficient to prevent cracks from opening to the surface under the influence of increased cavity pressures following a reactor coolant loop break accident.

To estimate the required depth of burial, a study was made of the distribution of rock stress between an unlined cavity and the surface for various cavity pressures. The stresses resulting from lithostatic and cavity pressure loads were calculated for both spherical and cylindrical cavities using the closed form solutions from elastic theory. These equations are recorded in Appendix I.

The distribution of tangential stress above a pressurized cylindrical cavity is shown in Figure 4-1. The net tangential stress, σ_θ, obtained by combining the lithostatic and internal pressure components, varies between the cavity and surface from tension to compression and back to tension. Due to the low and unpredictable tensile strength of most rock media, one should expect that cracks will open wherever a tensile stress exists. A crack extending between an unlined cavity and surface would allow direct flow to the atmosphere and constitute failure of the rock containment. This condition is assumed to exist if all points along the critical section directly above the cavity are in tension.

Calculations were made to determine the minimum depth of burial as a function of cavity pressure. The presence of tensile stresses at all points (i.e., $\sigma_{\theta_P} + \sigma_{\theta_\ell} < 0$) directly above the cavity as the criterion for containment. The results are presented in Figures 4-2 and 4-3 for cylindrical and spherical cavities, respectively.

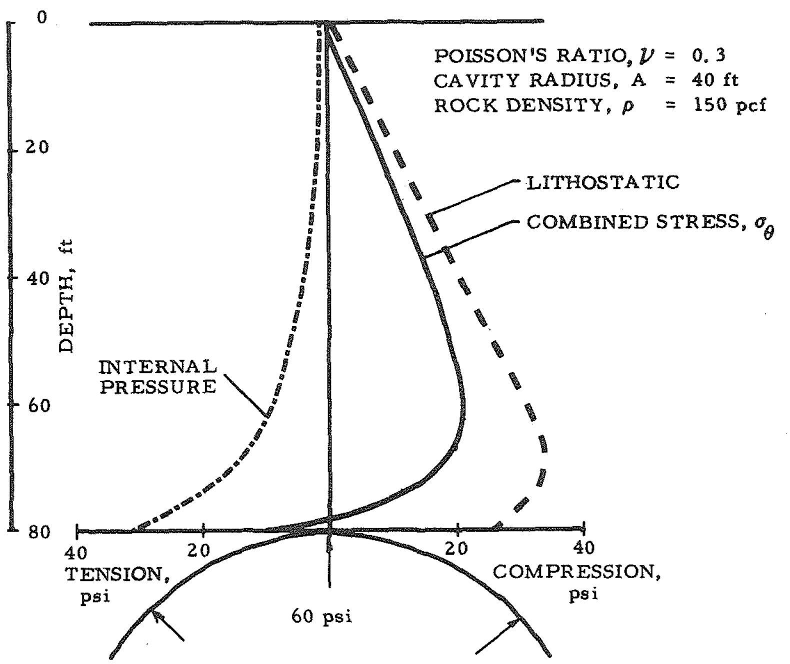


Figure 4-1. Stress Distribution Above Pressurized Rock Cavity (Cylindrical Case)

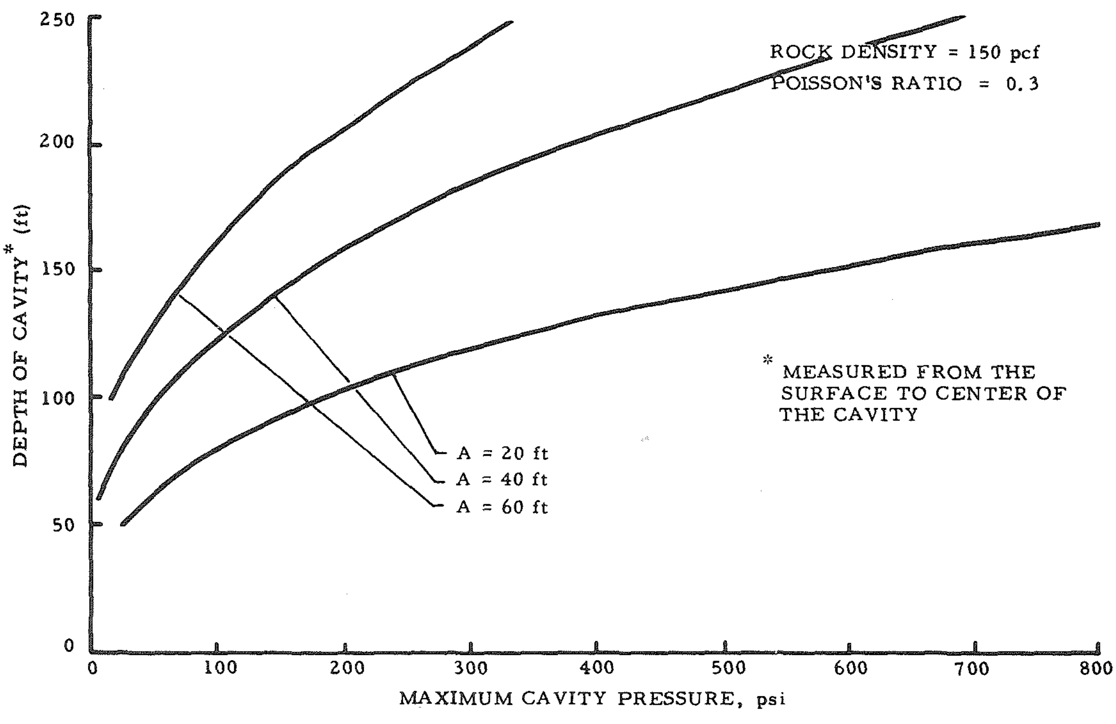


Figure 4-2. Cavity Depth vs Pressure (Cylindrical Case)

An important parameter influencing the required depth of burial is Poisson's ratio. This sensitivity is shown in Figure 4-4. For good rock, in situ horizontal and vertical stress measurements indicate an effective Poisson's ratio of between 0.3 and 0.4 even though laboratory tests on small specimens indicate lower values. Accordingly, a Poisson's ratio of 0.3 was used in Figures 4-2 and 4-3.

The change in stress with variations in cavity pressure is shown on Figure 4-5 for a 40 foot radius cylindrical cavity buried 120 feet. At cavity pressures of 93 psig or greater, a crack may extend to the surface as the stress is tensile at all points directly above the opening.

The size of the reactor cavities in good to excellent rock media identified in this study varied in span between 83 feet and 102 feet. Peak internal pressures were estimated to be between 74 psig and 58 psig, respectively. As can be seen from Figures 4-2, 4-3, and 4-4 a depth of cover of 80 to 130 feet should be adequate for structural containment without a margin of safety. The inclusion of a reasonable safety factor suggests depths of cover of 150 to 200 feet.

4.2 STRUCTURAL DESIGN

An investigation was made to determine the influence of site characteristics, system configuration, and structural parameters on the design of liners for underground power plant openings. Several liner concepts were examined. Special attention was given to the reactor systems gallery. The design sensitivity to geometry, rock quality, seismic loading, construction material properties and reactor accident environment were established.

4.2.1 Liner Concepts

The liner concepts shown in Figure 4-6 were considered. Included are liners constructed of reinforced concrete with a non-structural inner plate (leak protection), composite steel and concrete

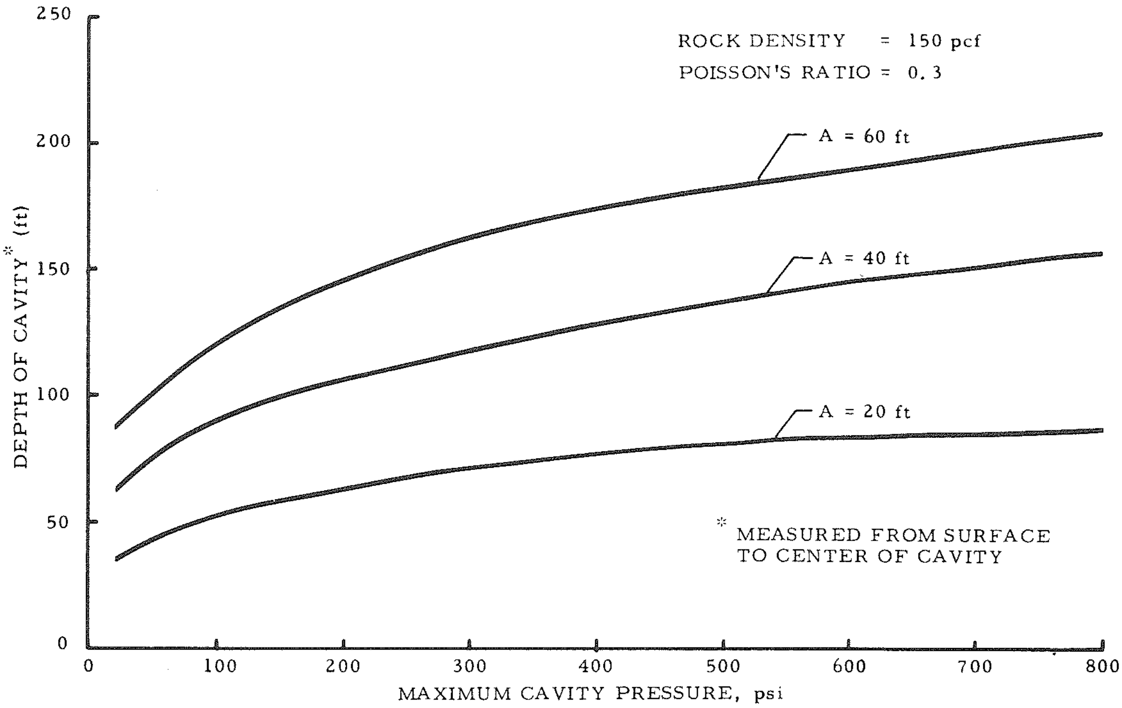


Figure 4-3. Cavity Depth vs Pressure (Spherical Case)

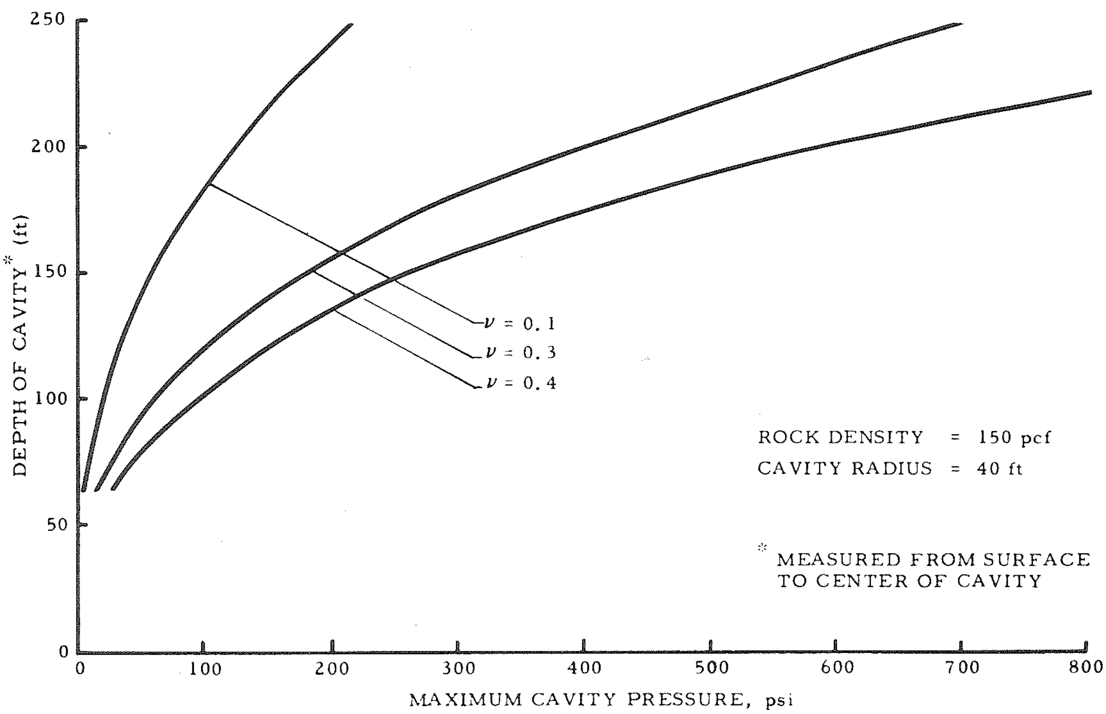


Figure 4-4. Cavity Depth vs Pressure (Cylindrical Case)

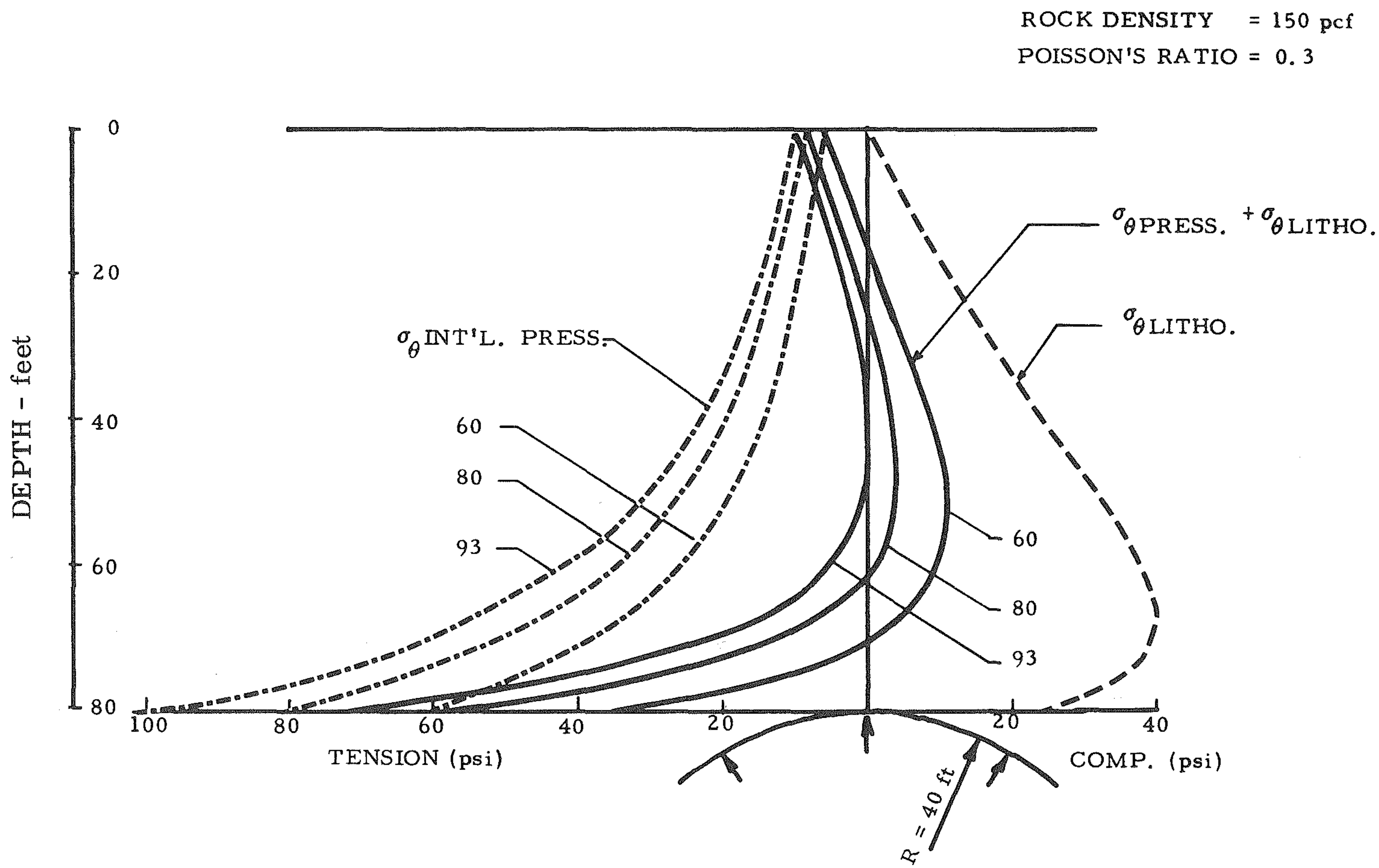


Figure 4-5. Rock Stress Distribution vs Cavity Pressure (Cylindrical Case)

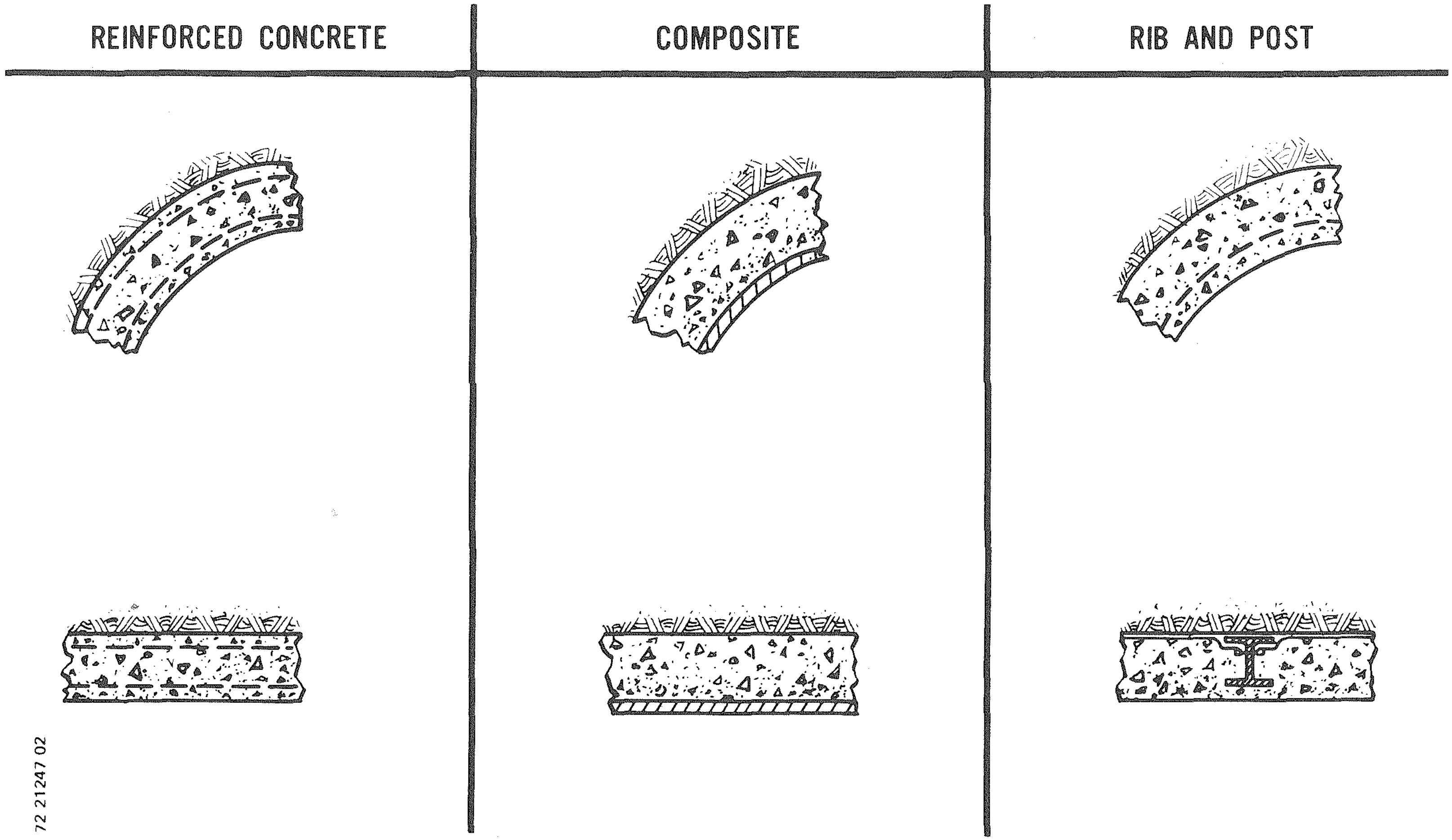


Figure 4-6. Liner Candidates

and steel ribs and posts. Reinforced concrete was found to be adequate for this application. The large size sections required for the rib and post design made it an impractical choice. The composite steel and concrete design was more than adequate from a structural standpoint.

Not explicitly identified in Figure 4-6 is a provision to allow water to drain around the structure to a sump volume. This provision is desirable to avoid designing the leak-tight underground structure as a pressure hull capable of withstanding a hydrostatic head. Drainage could be provided by installing a porous layer between the concrete and rock or installing drainage channels or pipes. The porous layer might consist of a thin layer of gravel, or porous concrete. The drainage channels might consist of drain tiles, perforated pipes, or a layer of corrugated steel adjacent to the rock. A sump storage volume would be provided for the drain water which would be expected to be contaminated following an accident. During normal operation this drainage would be monitored and pumped out of the facility. The influx of water around the reactor gallery could effectively prevent radioactive product seepage beyond the immediate area of the reactor gallery. The need for the water drainage provision including the calcification and potential clogging of drainage passages is recognized as a potential problem area that should be examined in future studies.

4.2.2 Analysis Methodology and Loads Determination

Working stress design principles as outlined in ACI Code 318-63, Reference 4-1, were selected for sizing the liners to be consistent with recommended practice (Reference 4-2) for containment structure design. The inner steel liner used in the containments was not considered to contribute to load carrying capability of the structure in keeping with the recommendations of Reference 4-2.

The loads assumed to be acting on the liner were:

a. Structure dead load.

b. Rock load dependent on rock quality.

c. Seismic amplification of above loads varying from 1.0 to 1.75 for roof segments and 0 to 0.75 for wall segments.

The pressure internal to the containment structure which might accompany a reactor accident was not included due to its transient nature. Final design calculations should take into account the time variation of seismic, temperature and internal pressure loadings, as well as site peculiar conditions. However, the foregoing assumptions are considered sufficiently accurate to identify the importance of various design parameters and are consistent with the characterization of a generalized site.

4.2.3 Liner Designs and Sensitivities

A horseshoe shaped containment, as shown in Figure 4-7, was chosen for parametric analyses. The results are shown in Figures 4-8 through 4-19. The minimum required arch thickness is the dependent variable on each graph. The several scales shown along the ordinate are described below. Figures 4-8 through 4-11 are for roof arches. They indicate the variation of required arch thickness as a function of rock quality, seismic load, rise to span ratio, and span in that order. Figures 4-12 through 4-15 provide the same information for the walls. The last three figures show the variation in thickness for different concrete and steel strengths and percentages of reinforcing steel. Figure 4-19 compares the thickness required of flat walls with parabolic arch walls. The thickness necessary to resist the rock seismic loads for flat walls make their use impractical even in excellent rock and lead to adoption of horseshoe-shaped structures consisting of parabolic arch segments.

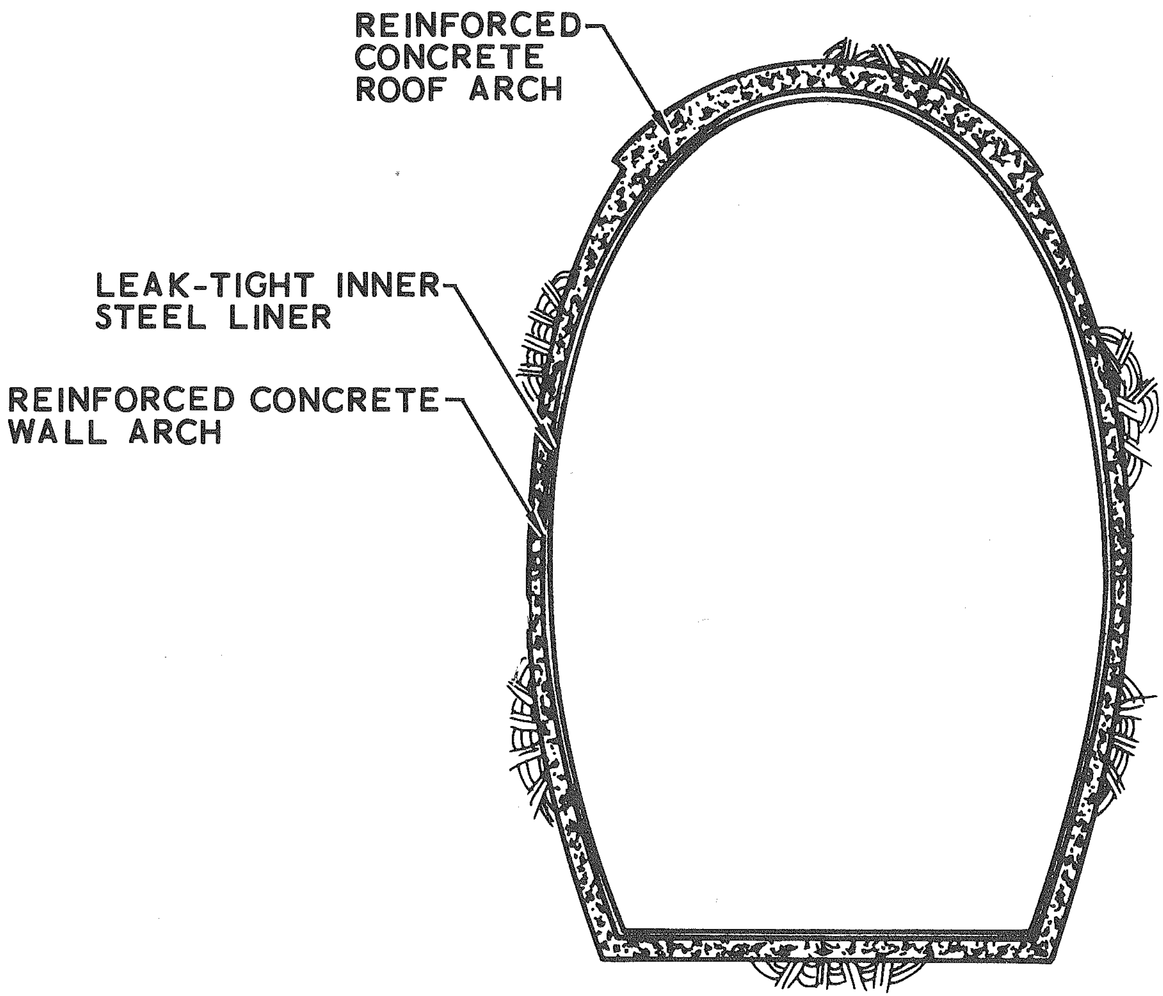


Figure 4-7. Reactor Containment Cross Section

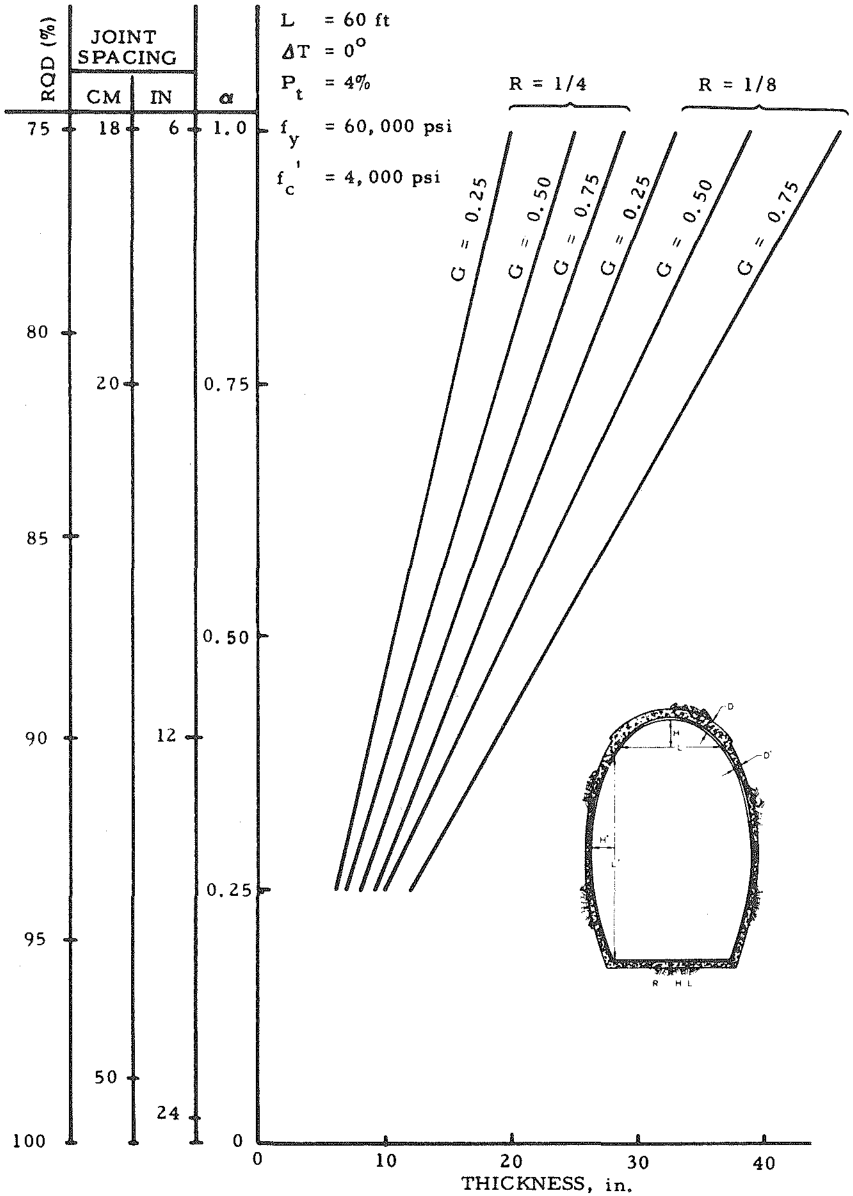


Figure 4-8. Parabolic Arch Roof α vs Thickness

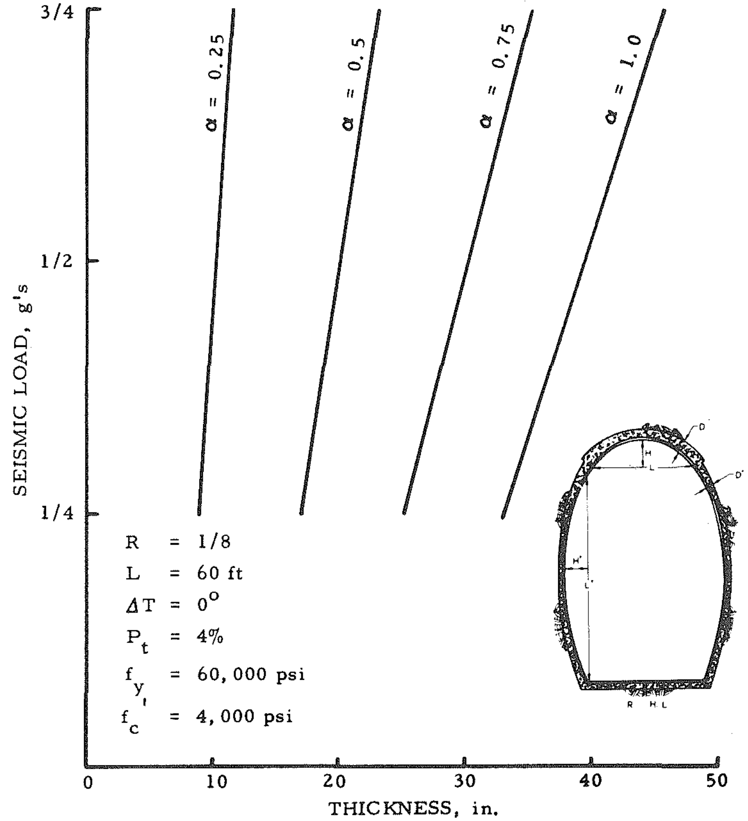


Figure 4-9. Parabolic Arch Roof Seismic Loading vs Thickness

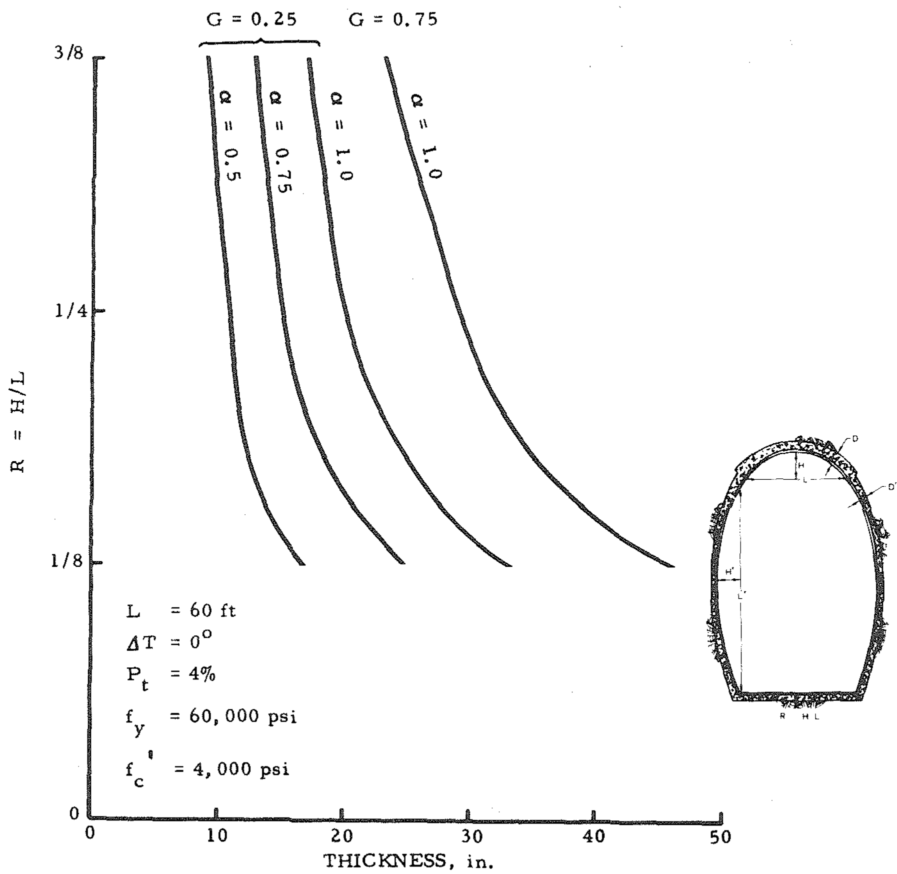


Figure 4-10. Parabolic Arch Roof Arch Rise vs Thickness

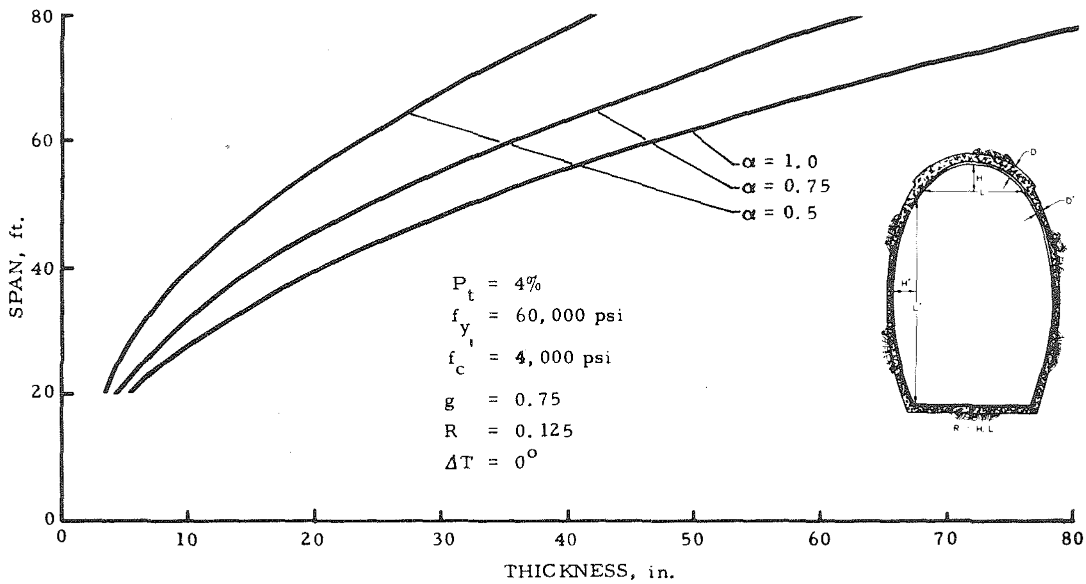


Figure 4-11. Parabolic Arch Roof Span vs Thickness

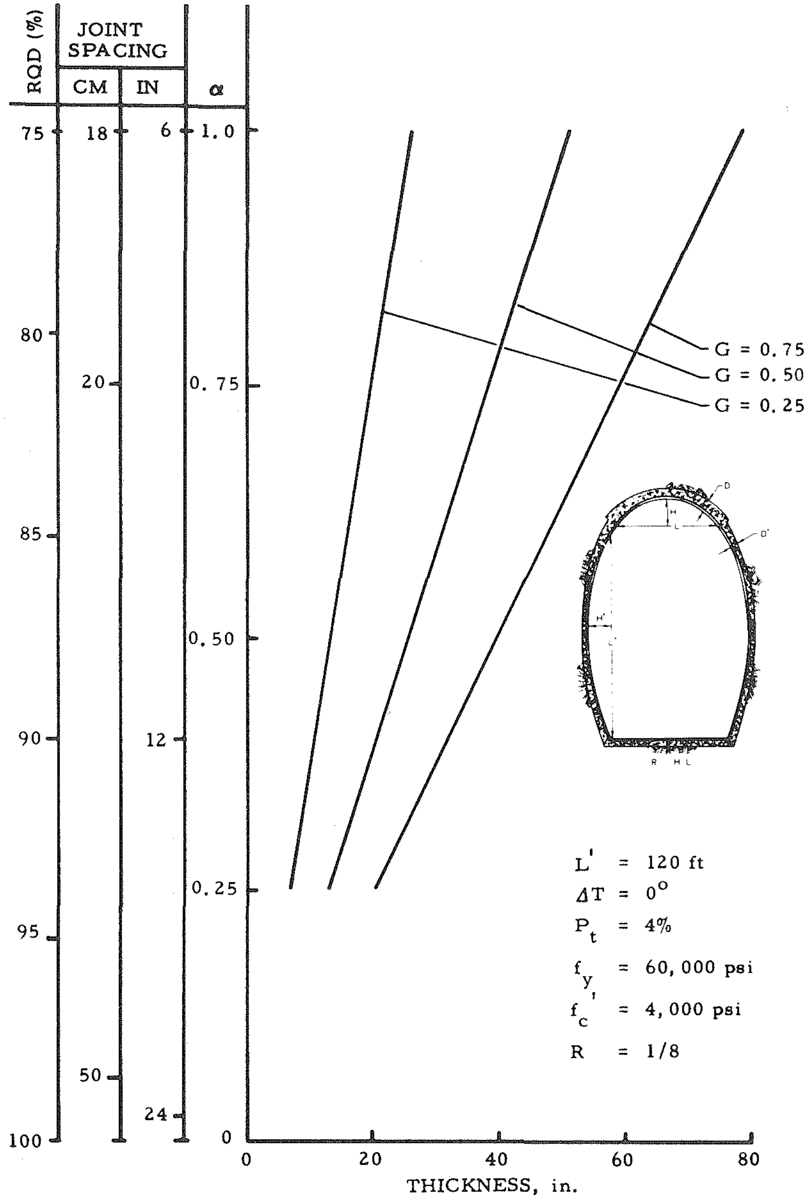


Figure 4-12. Parabolic Arch Wall α vs Thickness

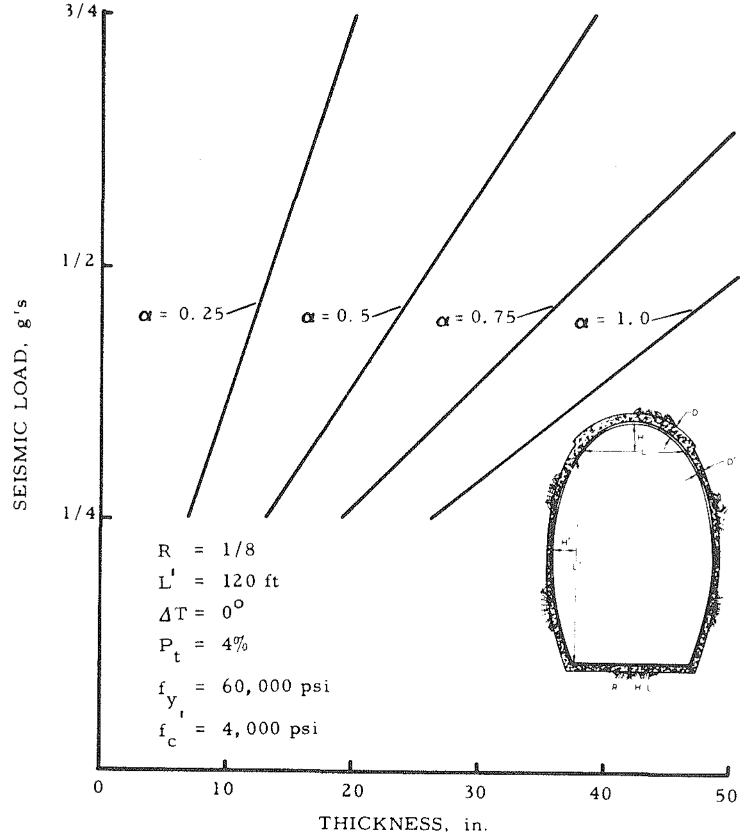


Figure 4-13. Parabolic Arch Wall Seismic Load vs Thickness

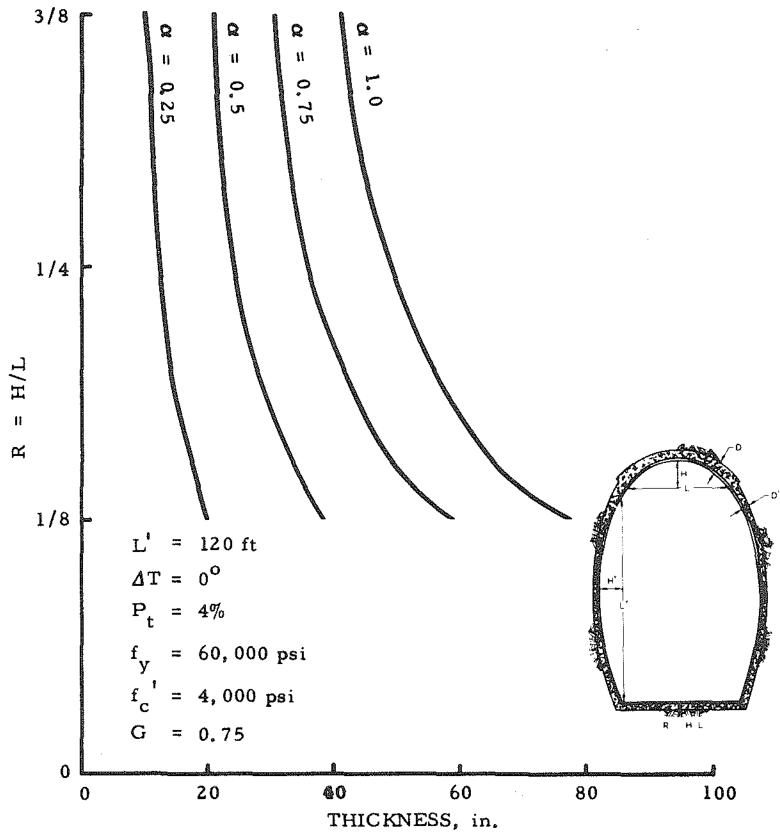


Figure 4-14. Parabolic Arch Wall Arch Rise vs Thickness

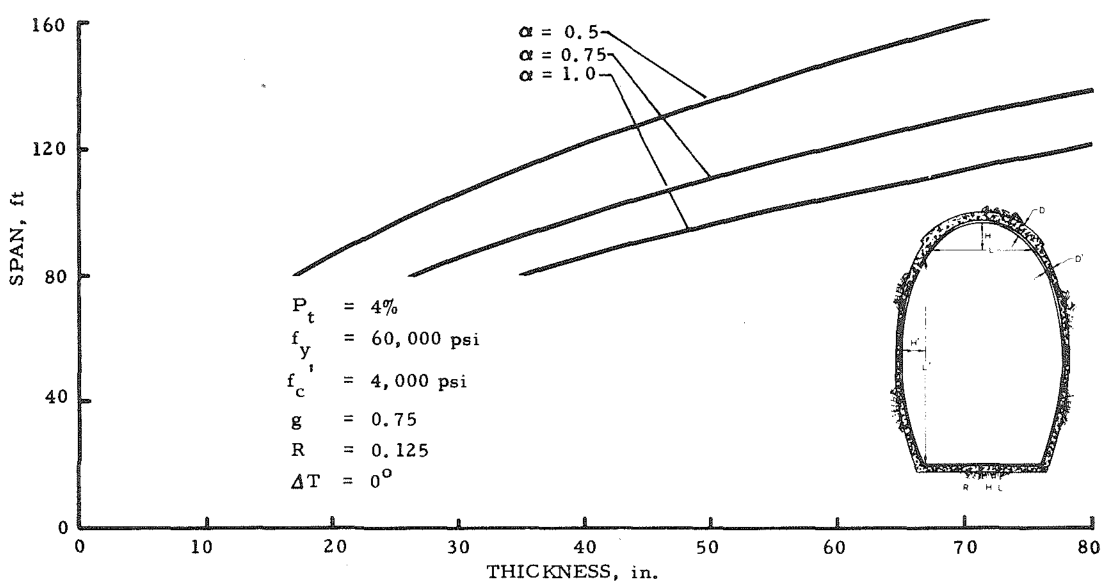


Figure 4-15. Parabolic Arch Wall Span vs Thickness

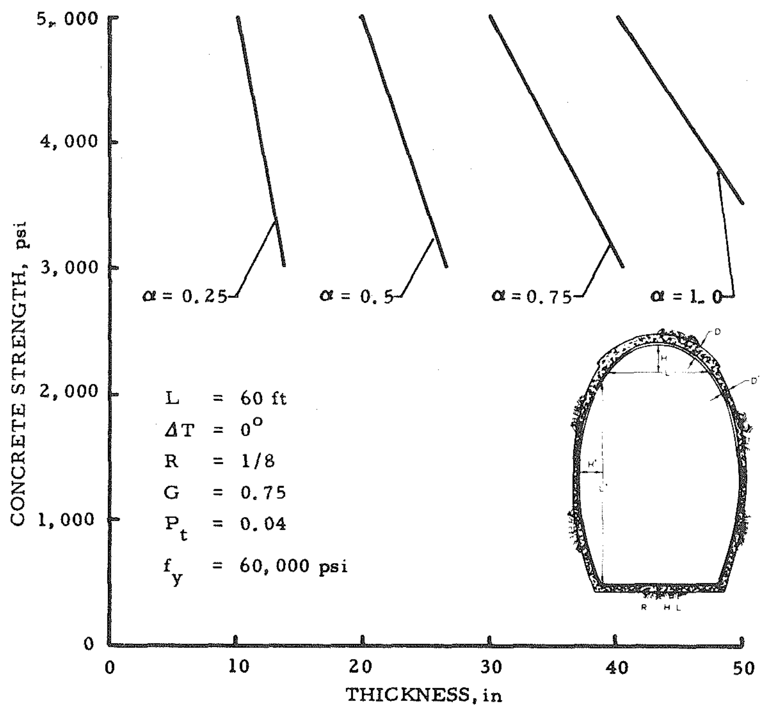


Figure 4-16. Parabolic Arch Roof
Concrete Strength vs Thickness

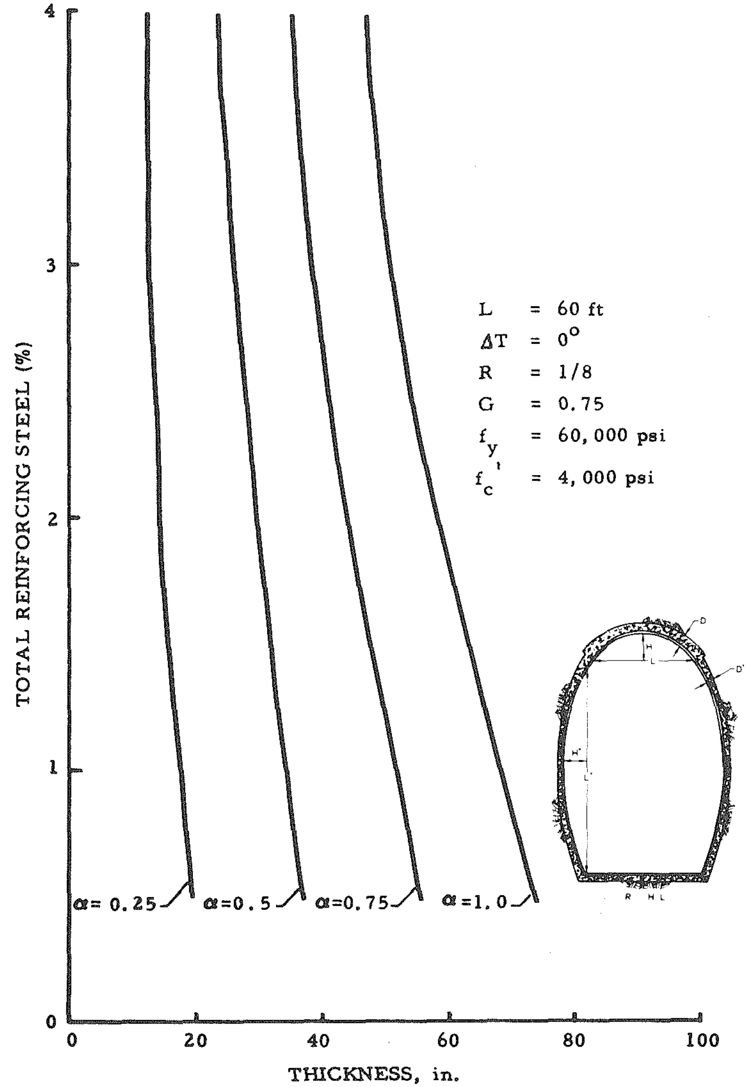


Figure 4-17. Parabolic Arch Roof
Reinforcing Percentage vs Thickness

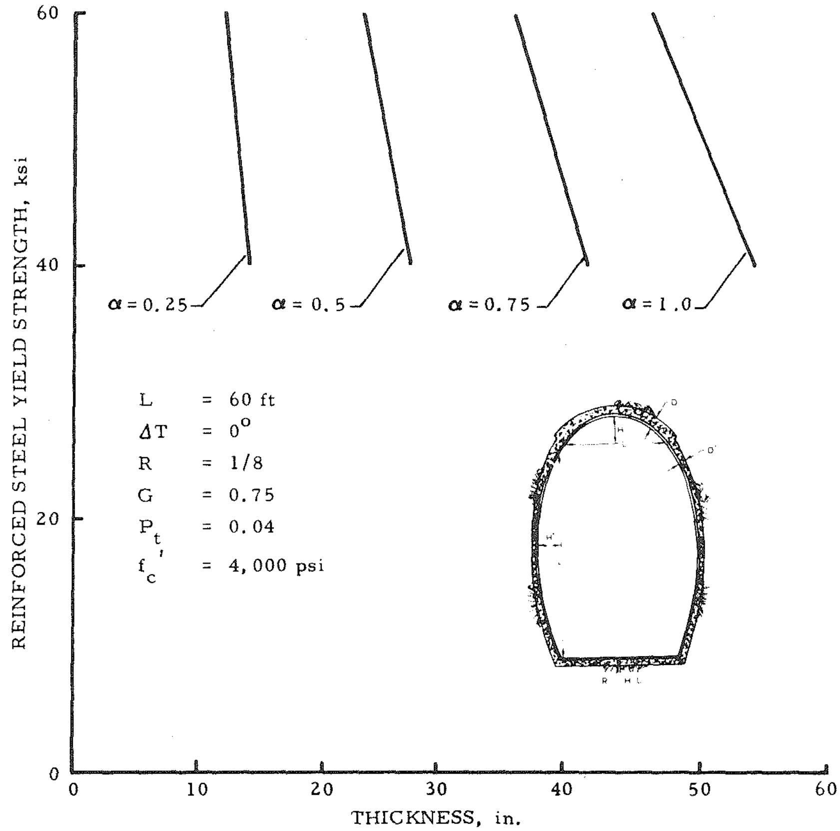


Figure 4-18. Parabolic Arch Roof Steel Yield Strength vs Thickness

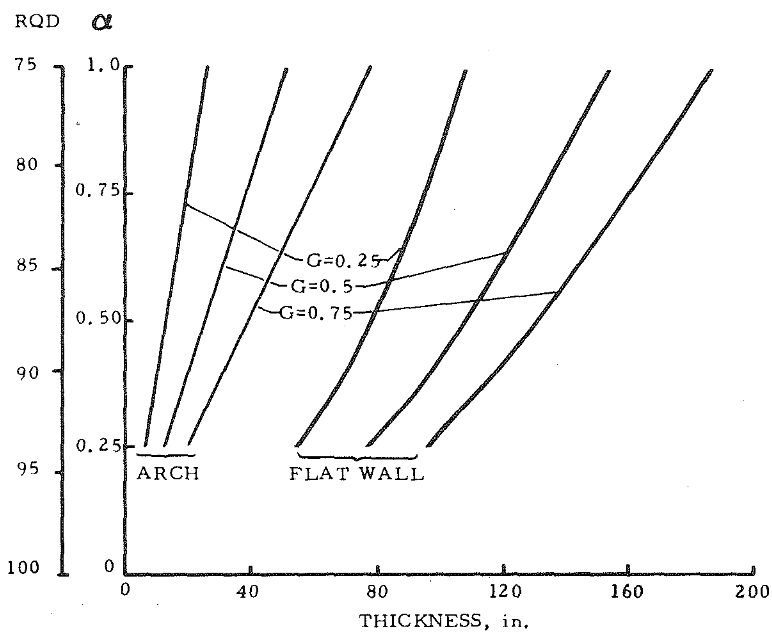


Figure 4-19. Arch vs Flat Wall

Figure 4-8 indicates the variation in arch thickness as a function of rock quality. The two scales used to characterize the media are α and RQD. The parameter, α, is taken from the Terzaghi classification system, Reference 4-3. It is a factor that when multiplied by the arch span determines the amount of rock the structure must be designed to support. In other words, a thickness of rock equal to α times the span must be supported by the structure. The joint spacing scale is self-explanatory. The RQD classification system, Reference 4-4, is based on examining core borings. An RQD is obtained by measuring the total length of all unweathered pieces of core greater than 4 inches and dividing by the total length of the core. The result is expressed as a percentage. A qualitative description of the rock for various RQD ratings is given in Table 4-1. The curves indicate that the required thickness of the arch is reasonable throughout the range of RQDs from 75 to 100 percent.

Table 4-1

ROCK QUALITY CLASSIFICATION

RQD (%)	Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Figure 4-9 indicates the variation in arch thickness as a function of seismic loading. Results are presented for a range from 25% g to 75% g and for rock qualities (α) of 0.25 to 1.0. From these results, it is apparent that the structural requirements are not particularly sensitive to variations in inertia loads within this range. This analysis does not account for differential movement of the rock such as might occur at a fault or major discontinuity during an earthquake. The latter would, of course, rupture any structure placed in direct contact with the rock.

Figure 4-10 shows the influence of the curvature of the arch. The results indicate rise to span ratios of 3/16 or greater are preferred. However, there is another factor to be considered in selecting the rise to span ratio which is not evident in Figure 4-10. As the ratio increases, the required structure thickness is reduced, but at the same time the opening becomes more bulbous requiring a larger excavation. The increase in excavation cost may offset the savings in liner costs. This was the case in the baseline structure designs discussed in Section 7.0 where a rise to span ratio of 1/8 was ultimately chosen.

Figure 4-11 illustrates the sensitivity of the structure thickness to span or opening size. It is seen that for spans of interest, the variation is nearly linear and the thickness is rather sensitive to this parameter. There are, of course, bounds on the opening size which are not apparent from the figure. Appendix II discusses this issue.

Figures 4-12 through 4-15 present the same information for arch walls as shown for the roof in Figures 4-8 through 4-11. The increased arch span makes the required thickness somewhat more sensitive to each of the parameters, but does not change the character of the sensitivity.

Figures 4-16, 4-17 and 4-18 show design sensitivity to concrete strength, steel yield strength and percentage of reinforcing steel. The designs do not appear particularly sensitive to material strengths or the amount of steel over normal ranges for these variables.

4.2.4 Thermal Stress

The foregoing parameter sensitivity curves did not include the effect of temperature induced stresses. Following a loss of coolant accident, the temperature within the containment could rise to between 275 and 325°F. The resulting stress distribution across the containment wall is shown in Figure 4-20 for a peak containment temperature of 325°F. These stress distributions are based on the assumption that the liner is prevented from lateral extension by the surrounding rock. This is a conservative assumption in that the in situ rock, while admittedly stiff, is nonetheless compressible. Extension of the arch through compressing the surrounding rock would relieve some of the stress.

Referring to Figure 4-20, 100 seconds after an accident, a stress of approximately 6000 psi is reached in the first inch or so of the liner. This stress level is in excess of the unconfined compressive strength of the concrete, assuming the use of 4000 psi, 28 day strength concrete. This may lead to some degradation of the first inch of thickness which is not sufficient to affect the liner's structural integrity. At later times, the rise in temperature and stress will progress toward the outer surface. However, the peak stress levels are reduced to within the unconfined compressive strength capability of the concrete. No adverse effect is expected, even though this idealized analysis indicates stresses in excess of the working stress allowables.

The foregoing results are based on a first order approximation of the thermal stress distribution. Since high stress levels are indicated, a more detailed analysis is justified. Numerical finite element analysis methods are well suited to problems of this sort, and their use should be considered in future investigations.

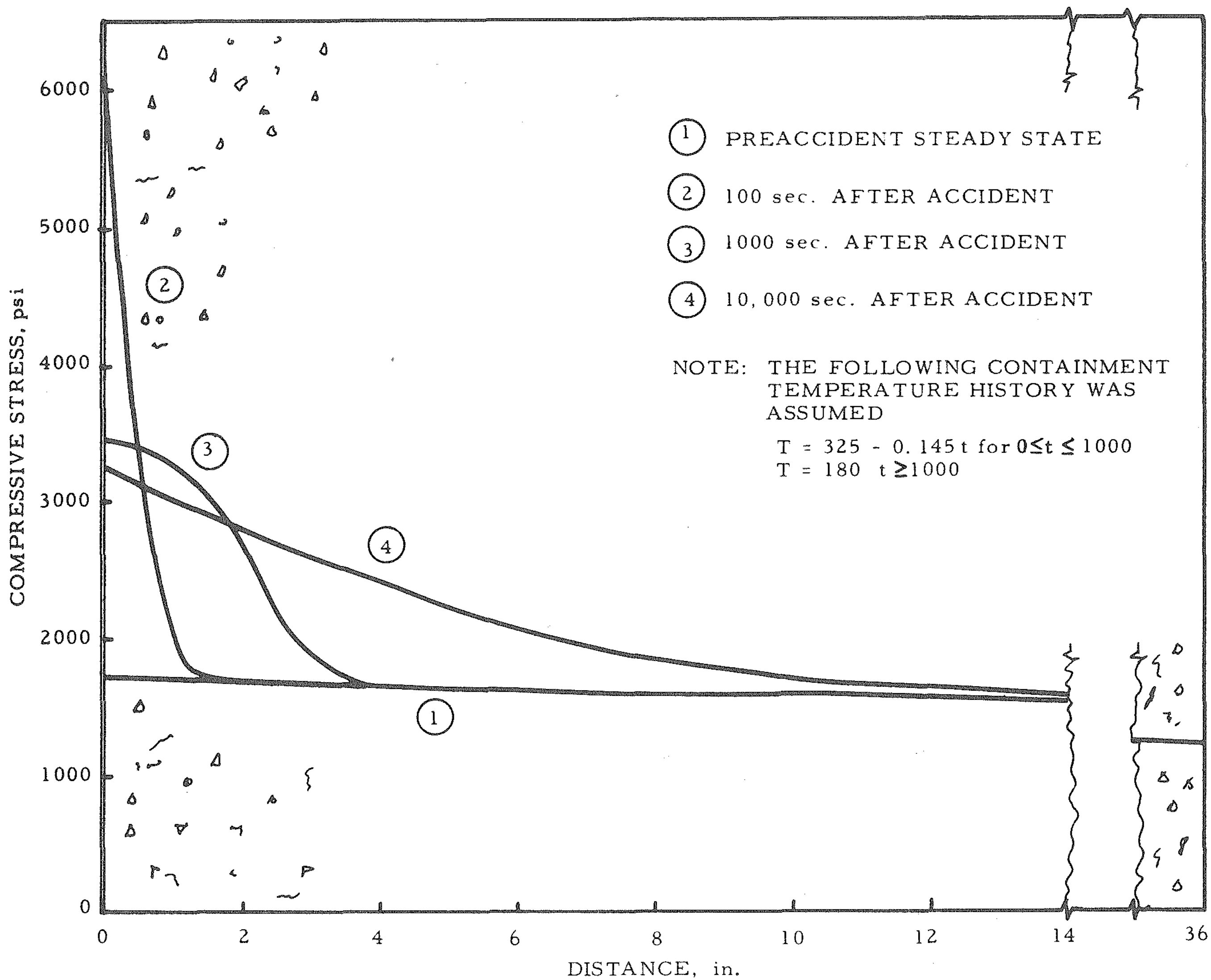


Figure 4-20. Thermal Stress Distribution Across Containment Wall

4.2.5 Baseline Structure Design

Liners were sized for each excavated gallery of the three plant configurations discussed in Section 3 to provide a basis for costing. In keeping with the AEC classification according to the degree of seismic protection to be provided, two classes of structures were considered as follows:

a. Class I--Those structures required to prevent uncontrolled release of radioactivity and those necessary to maintain the plant in a safe shutdown condition.

b. Class II--Those structures required for continuity of power generation but not essential to safe shutdown and isolation of the reactor and whose failure could not result in the release of substantial amounts of radioactivity.

The dimensions and class assignments for each gallery are summarized in Appendix VI. The reactor gallery, nuclear auxiliary gallery, and miscellaneous-control gallery were designated Class I and the turbine-generator gallery Class II. For designation of the smaller excavations see Appendix VI. Class I and Class II structures were designed to withstand earthquake inertia loadings of 50% g and 25% g, respectively, in a horizontal direction and 2/3 of these levels vertically. The other baseline design parameters were:

Rock Quality, RQD	80 to 85
Rock Load Factor, α	0.5
Concrete Strength, f_c'	4000 psi
Steel Yield Strength, f_y	60,000 psi
Reinforcing Percentage, P_t	1%
Arch Rise to Span Ratio, R	1/8

The cost of excavation and structural liners for the baseline plant layouts was calculated and is provided together with other cost information in Section 7.0.

4.2.6 Structure Summary

The structures for underground plants in good to excellent rock media require reinforced concrete wall thicknesses up to five feet for the larger openings. Siting in poor rock media will result in very thick walls (up to 25 feet) and will probably result in construction difficulties for large span galleries. The use of horseshoe shaped openings is preferred rather than the more common arch roof and flat walled construction found in most hydroelectric plants. The requirement to design for earthquake inertia loads leads to this selection. The inertia loads are not the only possible earthquake effect influencing liner design. Rock fracture and differential movement, should it occur, would present a problem. A crushable backpacking material might be used if it were necessary to accommodate such conditions. The likelihood of a virgin fault opening where a fault had not previously existed is believed to be remote. With proper siting precautions the need to design for differential rock motion can be avoided. Finally, the temperature differentials following an accident might result in high stresses and, while these are not expected to affect the liner's structural integrity, they should be examined more thoroughly.

4.3 SEISMIC ANALYSES

A careful evaluation of the seismic implications of underground nuclear power plant siting was hoped to be included as part of this initial investigation. Unfortunately, the limited study resources and schedule did not permit this effort to be completed. Such an investigation is recommended as part of future efforts. The following discussion summarizes the results of preliminary thoughts on the subject but are not based on quantitative analyses.

Protection from earthquake loadings is one of the most difficult problems in nuclear power plant design and siting. Underground siting does not appear to provide a total solution to this problem although there are several potential advantages that may make the problem more tractable. The greater containment provided by underground construction suggests the consequences of damage following an earthquake may be less severe. This does not in itself reduce the probability of an accident. The ability to more rigidly support major pieces of equipment from the walls or roof might also reduce the amplification of seismic loads expected for surface plants. In general, for both surface or underground plants protection from earthquakes may be obtained by selecting a site where the earthquake loads are reduced, through special design of equipment to resist earthquake loads, by special provision to reduce amplification of external loads, and/or by inserting some type of isolation mechanism to attenuate the external loads to acceptable levels. All three of these approaches are employed in present nuclear plants to varying degrees.

4.3.1 Seismic Protection--Motion Attenuation

The earthquake loading at any site is quite dependent on the specific site. Economic factors will tend to limit selection of underground sites to reasonably competent rock masses. The motions experienced by an underground plant should then be unaffected by the amplifications sometimes encountered at surface sites placed on soft media. The underground site will also be free from the problems associated with soil liquefaction.

It has been suggested that the motions occurring at some depth below the surface may be less than those at the surface due to the attenuation of surface wave amplitudes away from the surface. However, since the wave lengths associated with earthquake frequencies are much longer than the probable depths of burial for underground plants, significant attenuations may not be observed. Furthermore, the complex structure of surface waves may include some components that peak below the surface. This problem was not quantitatively investigated in the present study.

4.3.2 Seismic Protection--Fault Identification

In the design of surface nuclear power plants great care and expense is taken to identify nearby faults. These faults are frequently obscured by surface layers or obstructions. Large differential motions across a fault running through a nuclear plant presents a near impossible design task and such faulted sites must be avoided. Construction of an underground plant would be preceded by similar extensive geologic and geophysical investigations. The excavation of the underground plant would provide for the detailed direct inspection of the geology of the plant site which cannot be accomplished for surface sites. If an unknown fault zone were uncovered, the site could be abandoned. Where the inspection of the rock mass did not reveal fault zones, it is thought safe to assert that large differential motions from future earthquakes will not occur at the plant site although uniform gross motions may be experienced.

4.3.3 Seismic Protection--Reduced Amplification and Isolation

The large mass of critical reactor system equipment and the necessity to maintain the integrity of interconnecting lines presents a severe shock isolation problem for many reactor systems. This is true for both underground and surface plants. In some surface plants the amplification of the base motions due to flexure of the equipment supports and plant structure can be a factor of two or more. These application factors may be reduced by providing more rigid supports (increasing the effective frequency of the supporting structure). The design of such supports may be simplified in underground plants due to the greater access to the base rock of the walls or roof. Some reactor systems such as the high temperature gas cooled reactors (HTGR) may inherently be more resistant to earthquake accelerations, particularly if incorporated in an underground configuration.

An alternate to stiffening the ties between the plant equipment and the base rock is the introduction of an isolation system (decreasing the frequency of the supporting structure). Underground siting of the light water reactor systems considered in the present study does not appear to simplify the shock isolation required for seismic protection. It is possible that a detailed engineering design of shock mounts for individual components or mounts to support a large platform containing the reactor steam system could make use of the sidewalls and roof for support that would not be practical in surface plants. Such designs have not been developed.

SECTION 5

CONTAINMENT ANALYSES

An attractive feature of underground nuclear power plant siting is the potential improvement in the containment of radioactive materials in the event of an accident. Present surface nuclear plants must be sited at large distances from population centers. The separation between a 1000 Mwe plant site and an actual or future population center of 25,000 or more people might typically be of the order of 15 to 20 miles. Other separation distances apply to lesser population levels. An exclusion area in the immediate vicinity of the surface plant must be defined within which the public can be excluded and positive control for possible evacuation can be maintained. These siting requirements are derived from conservatively safe estimates of the worst conditions that could result following a maximum credible accident at the power plant coupled with what are thought to be conservatively low permissible radiation dose levels that might be imposed upon the public. These large separation distances effectively prohibit metropolitan siting of nuclear plants near load centers and exclude large areas from consideration as possible plant sites.

The large separation distances are dictated by postulated accident conditions and do not relate to normal operations. Siting of a plant underground in some media could result in near total containment of radioactive materials following an accident and significantly reduce the separation distances from miles to fractions of a mile, feet, or essentially zero. The improved containment would be provided by the earth cover over the plant. Several idealized calculations were performed to explore the effectiveness of this containment.

5.1 IDEALIZED MODEL RESULTS

The reactor galleries described in Sections 3 and 4 are provided with a reinforced concrete and steel liner. This liner would be constructed to leak tight specifications similar to the containment structures of surface plants. For purposes of analysis it was decided to initially consider an unlined cavity and examine the containment that might be provided by typical rock media. The presence of the concrete and steel inner liner makes this assumption extremely conservative. To the extent that this liner is identical with the containment structure of a surface plant, the containment provided by the rock medium is an improvement that will decrease the required separation distances. The improved containment can also be viewed as an additional margin to compensate for unforeseen adverse performance of present safety systems. The barrier provided by the rock might also be considered as an additional passive safety system.

For purposes of analysis an unlined spherical cavity with a radius of 17 m (approximate volume of the PWR reactor gallery) was adopted. This cavity is assumed to be buried in a dry uniform medium characterized by a porosity and a permeability. Two types of media are considered corresponding to a granitic medium with low porosity (perhaps 0.1%) and a sandstone or limestone medium with a high porosity (perhaps 15 - 20%). The permeability of both media is postulated to be between 1 and 10 millidarcies. (A discussion of rock porosity and permeability is contained in Section 5.2 and Appendix IV.)

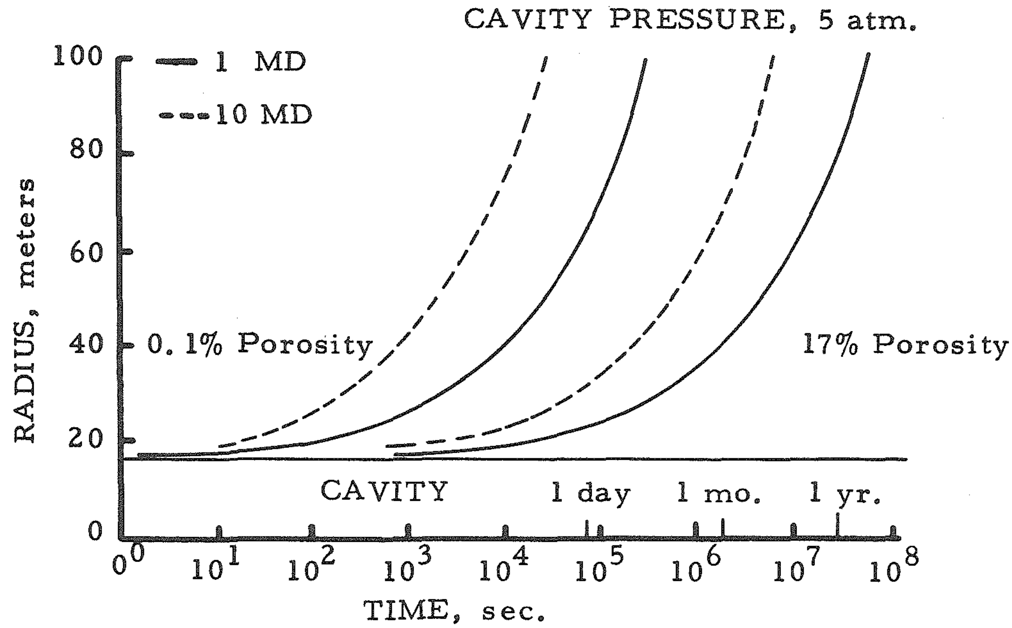
The volume of the 17 m cavity is approximately 2×10^4 m^3 and the surface area is 4×10^3 m^2. The pore volume in a sphere with twice the cavity radius is equal to the cavity volume if the porosity is only 14%. The internal surface area of a medium-fine sandstone such as Bradford Sandstone is ~5000 acres/acre-ft; and may reach 30,000 acres/acre-ft in a fine sandstone. Hence, a seepage front at 34 meters will have filtered over $\sim 2 \times 10^9$ m^2 of relatively cool rock surface. The heat capacity of the rock is also very large, and will cause the steam diffusing into the rock to condense with a drop in pressure. The particulate materials and the halogens will also be cooled and adsorbed in the pore volume very near the cavity. Only the noble gases, e.g. Kr-85, will persist in a form that can move large distances through the dry rock medium.

To obtain an idea of the seepage rates from the reactor cavity, some simplifying assumptions were necessary. A steady state isothermal flow is postulated between a spherical cavity and the seepage front. The pressure is assumed to drop from the cavity pressure (3 or 5 atmospheres) at the cavity surface to 1 atmosphere at the seepage front. Figure 5-1 shows the position of the front as a function of time for several parametric values for the cavity pressure, rock porosity, and permeability. It is seen that the time required for the front to move a distance of 60-70 meters (approximate minimum depth of burial from Section 4) is very long. Most of the radioactive gasses that are produced in a reactor have half-lives much less than these times and will decay. I-131, which is probably the most prominent isotope in establishing surface plant separations, has a half-life of 8.1 days and would experience significant decay even if it were not trapped within the rock pores. Table 5-1 lists several radioactive half-lives for fission products. This calculational model greatly overstates the leakage problem. The isothermal assumption assumes the rock is at the same temperature as the steam. In reality, the steam temperature and pressure will drop sharply once it moves away from the cavity and the relatively cool rock will cause the steam to condense and the pressure to drop. The model further does not allow for condensation of the steam in the reactor cavity and reduction of the cavity pressure. The cavity pressure will drop quickly as the emergency cooling systems condense the steam in the reactor cavity. Also, the model does not account for the volume of material that is lost from the cavity through seepage. As mentioned previously, the pore volume already equals the cavity volume at only 2 cavity radii for a material with 14% porosity. The isothermal flow rates predicted from the cavity imply that the cavity will be depressurized by volumetric loss before the seepage front reaches the surface.

Table 5-1

FISSION PRODUCT HALF-LIVES

Product	Half-Life
Kr-85	10.4 yr
Kr-87	78 min
Kr-88	2.8 hr
Xe-133	5.27 days
Xe-135	9.2 hr
Xe-138	17 min
I-131	8.1 days



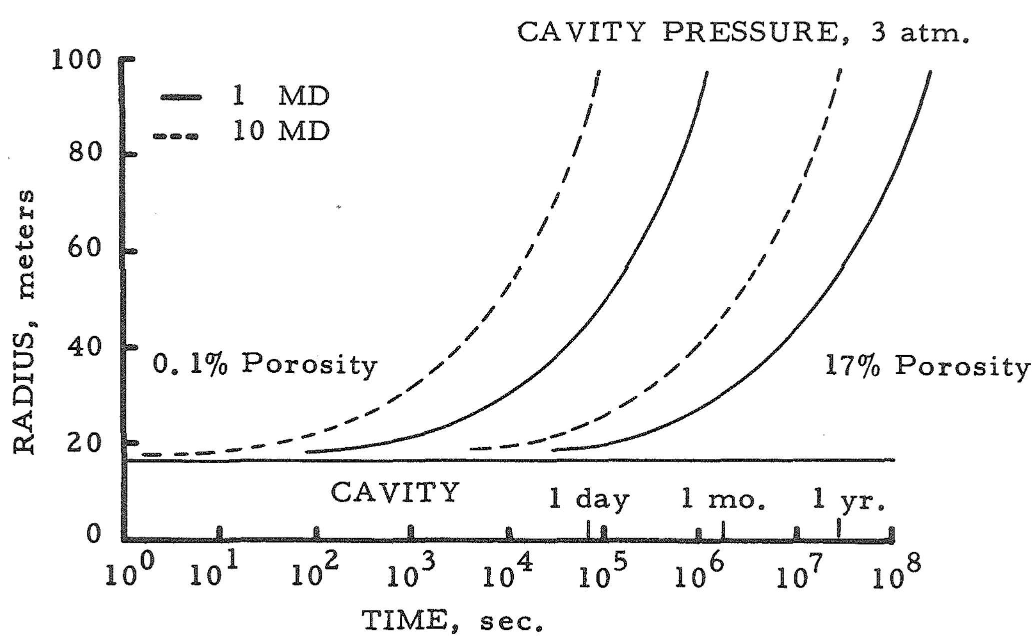


Figure 5-1. Seepage Front Position

A drop in the cavity pressure will affect the seepage by effectively removing the driving force. When the cavity pressure drops to 1 atmosphere, seepage will essentially stop. The steam and radioactive material will then be trapped in the rock, spread over a region roughly extending from the cavity to the seepage front.

The pressure time history in the reactor cavity following a maximum credible accident is determined by the rate of release of water or steam through the postulated break in the primary loop, the extent of chemical interactions (e.g. zirconium-water), the energy generated from the core, the volume and heat capacity of the containment structure and the emergency cooling systems designed to minimize the temperature and pressure. Although the volume of the underground reactor cavity may be less than that typically found in surface plants and peak temperatures and pressures larger, the duration of the internal pressure pulse should be very similar (assuming the same emergency systems). This can be seen by noting that the primary mechanism for reducing the cavity temperature and pressure is through injecting large quantities of water rather than by heat loss through the air or rock. If the underground systems inject a comparable amount of water, the end result should be about the same. The pressure-time history for the underground cavity was estimated as shown in Figure 5-2. The duration is seen to be approximately 10^3 seconds or 17 minutes. The seepage driving force will then terminate after only 10^3 seconds which is well before the front has reached the surface. The distances the front has moved can be read directly from Figure 5-1 or 5-2. It is seen that for many media, the seepage front will have progressed only a few meters from the unlined cavity. A large porosity is preferred because of the large storage capacity in the rock. This model implies that no material will be released to the atmosphere and complete containment will be achieved.

Further advance of the radioactivity beyond the seepage front can take place only by diffusion of the radioactive gases through the stagnant air in the pores of the enclosing rock. In this case, transport is caused by concentration gradients rather than pressure gradients, and is very slow in comparison with the connective flow rates considered above. Diffusion is a "random-walk" process, and it is characteristic of these that they lead to probability distributions of the form

$$P(x)\ dx \sim e^{-x^2/2\xi^2}\ dx \quad \text{(gaussian)}$$

where P (x) dx is the probability that the diffusing particle will have moved into the interval between x and x + dx after elapsed time t. Time enters through the rms displacement

$$\xi = \sqrt{2Dt}$$

where D is the diffusion coefficient, which is a function of pressure, temperature, and concentration of the diffusing gases. In a fine-grained rock, an additional factor should be applied for the effects of pore geometry (tortuosity), effectively reducing D. A reasonable value for D (neglecting tortuosity) is $\sim 0.1\ cm^2/sec$. If we take $t = 3 \times 10^7$ seconds ($\approx$ 1 year), we have

$$\xi \approx 25 \text{ meters}$$

This means, for example, that an initially well-defined concentration front will have smeared over ~25 meters after 1 year. It therefore appears that krypton-85 (half-life 10.4 years) could eventually reach the surface, in an unsaturated rock, but at a negligible discharge rate into the atmosphere.

The preceding arguments apply to a dry medium. Many, if not most, rock media are expected to contain pore water. As mentioned in Section 4, provision is made to allow this water to drain around the

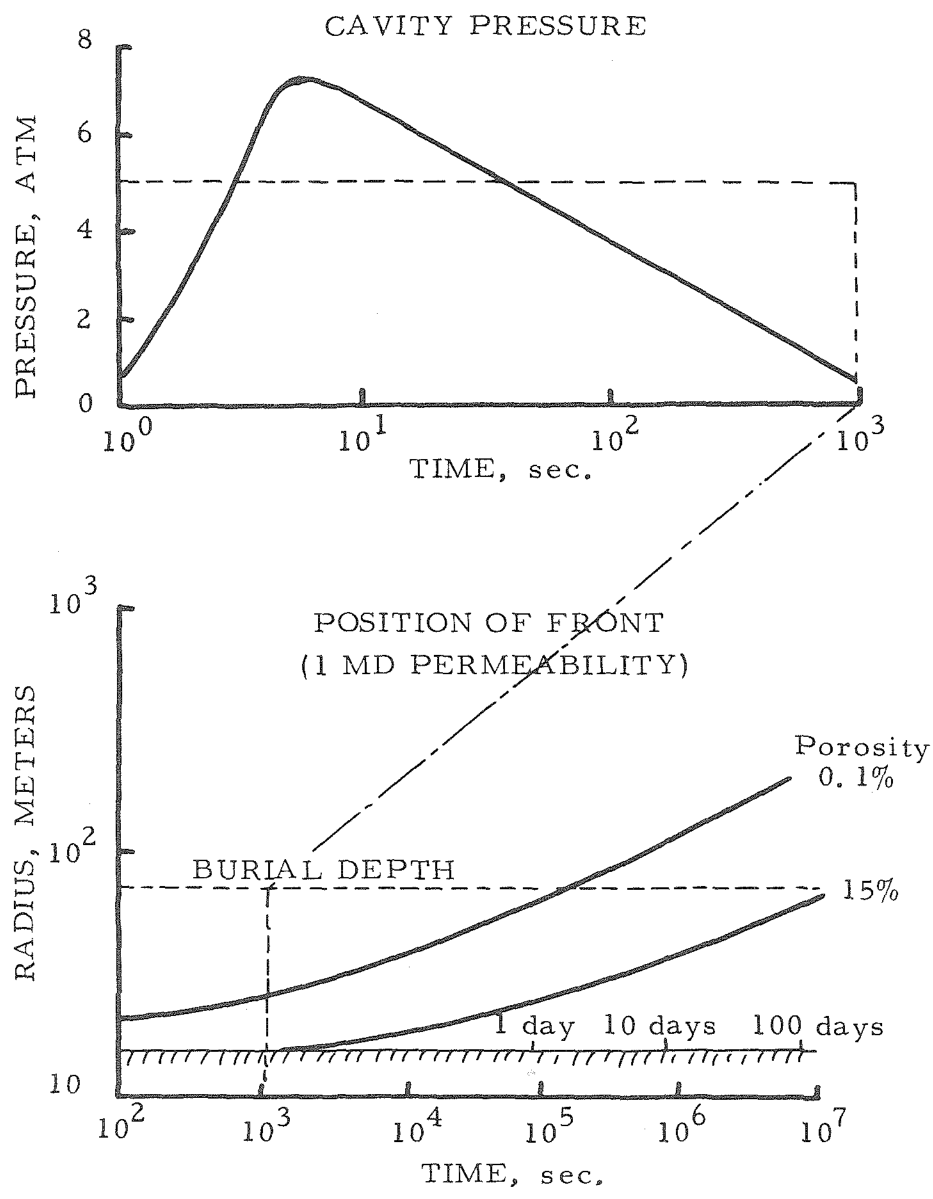


Figure 5-2. Cavity Pressure and Seepage Front Position

cavity liners, to a sump. The presence of water in a fine grained rock will decrease the permeability and increase the pressure required to force a gas through the rock. The pressure increase is required due to the need to displace capillary bound water from the pores. Clay materials in the pores also swell and significantly reduce permeability when wet. The flow in toward the cavity sump will also tend to leach out any material that does escape the containment liner.

5.2 POROSITY AND PERMEABILITY OF ROCKS

The previous section assumed a uniform dry rock medium characterized by a single porosity and a single permeability. A "granite" medium with 0.1% porosity and 1-10 millidarcy permeability and a "sandstone/limestone" medium with 15 - 20% porosity and 1 - 10 millidarcy permeability were considered. Naturally occurring rock media are not uniform and only approximate these values in their bulk behavior. This section addresses the factors that lead to the bulk porosity and permeability and the variations that might be expected at typical sites.

The movement of fluids in rocks has been studied extensively by the petroleum industry, among others. Two physical parameters of rock (or soil, sand, etc.) are of importance here: porosity and permeability. Porosity is a measure of the capacity to store fluids, and permeability defines the capacity to transmit fluids. A permeable rock must have porosity, but porosity does not imply permeability. In order for a rock to be permeable, it must have interconnecting pores, of supercapillary size. Pumice is porous (up to 90% or more void space) but not permeable, since the pores are not interconnected. Shales and clays are also porous, but generally impermeable because the pores are extremely fine and filled with water. The roof rock in oil and gas pools is generally clay or clay-containing, such as shales, sandy shales, or shaly sandstones. Although not strictly impervious, these rocks prevent upward seepage of gas and oil because the pressures available in the oil reservoirs are insufficient to displace capillary water from the extremely fine pores of the confining strata. It should be noted that clays and shales comprise roughly 80% of all sedimentary rocks.

Two types of porosity are distinguished: primary, or intergranular, as in a sand or sandstone; and secondary porosity. Secondary porosity is any form of void space which bears no direct relationship to the grains of the rock, and is also designated variously as intermediate, fracture, or induced porosity. In brittle rocks, fractures and joints are common forms of secondary porosity. These joints occasionally enable even such intrinsically dense, non-porous rocks as granites to serve as productive reservoirs. Fractures and solution cavities and channels are important forms of porosity in carbonate rocks, the pores ranging from fractions of a millimeter at one extreme to large caverns at the other. Porosity and permeability are variable functions of position, even within a nominally homogeneous rock.

Figure 5-3 (Ref. 5-1) shows measured porosities and permeabilities for about 500 small core samples of a fine, uniform-grained sandstone from Pennsylvania. Permeabilities are conventionally measured parallel to the bedding planes, using oven-dried specimens and dry air. Vertical permeabilities across bedding planes which are of primary interest for underground power plants are usually lower. The use of dry air avoids problems introduced by the swelling of clay minerals typically found in reservoir rocks which decreases permeability. Where moisture is found in the rock the permeability will be reduced.

It should be emphasized that the permeability of a sedimentary rock is generally greater in the horizontal direction, as a result of arrangement and packing during deposition. In stratified rock the permeability will vary from one stratum to the next.

The following quotation is taken from Reference 5-2.

> "Most of the oil and gas fields of the world are in sedimentary rocks, and most sedimentary rocks contain bedding planes or partings on which commonly,

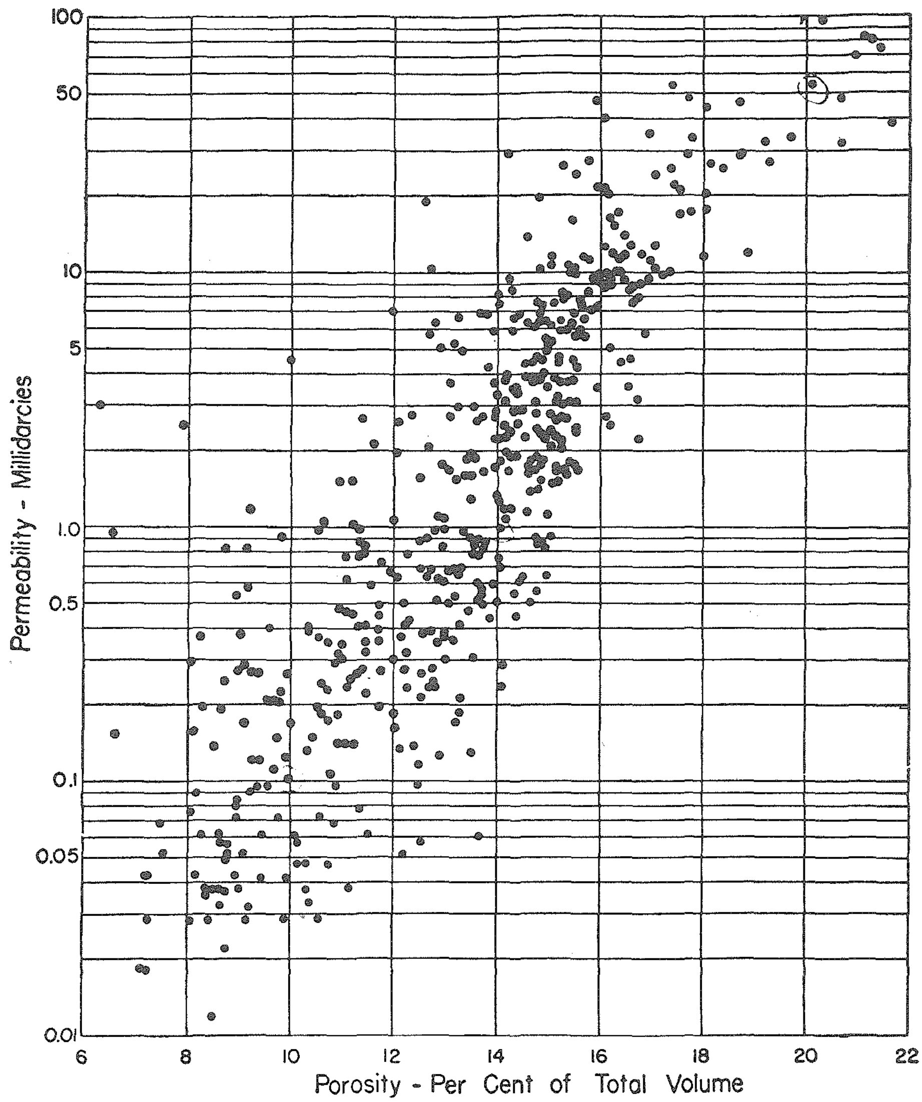


Figure 5-3. Porosity and Permeability Correlation in Bradford Sandstone (Pennsylvania)

> although not invariably, there is a film or selvage of clay that may be an effective barrier to the free movement of oil or gas from the rock on one side of the bedding plane to the rock on the other. Even where there is no clay film there may be induration or cementation at the bedding plane, which will reduce the permeability. These bedding planes are believed to be responsible for the observed phenomenon of very low vertical permeability as manifested by the behavior of wells even though the measurements in the laboratory on short cylinders or "plugs" of the reservoir rock may have shown high permeability values. The samples tested in the laboratory rarely include bedding planes. In the process of coring to secure samples, the rock tends to break along clay partings and bedding planes so that the fragments that are available for measurements do not include the barriers to transverse (i.e., vertical) movement of fluids."

Permeability can be even more variable in a rock having fracture porosity. Wells (oil or water) drilled into rock having only fracture porosity depend for their success upon intersecting a network of fractures through which the fluid may move freely. Production from such wells is very erratic, and dry holes often occur adjacent to productive wells. Great effort has gone into development of techniques to increase fracture porosity, such as "shooting" wells with high explosives, pumping of acid into limestone reservoirs, hydraulic fracturing, etc., and the recent Rulison and Gasbuggy nuclear gas-stimulation tests (Project Plowshare). Such efforts are, of course, inappropriate at underground nuclear power plant sites where low permeability is desirable. The existence of these efforts suggests that sites with low permeabilities are not atypical.

The principal oil and gas reservoir rocks are sandstones (15% of all sedimentary rocks) and limestone (5%). Practically all sandstones are interbedded with clays and shales, interrupting their permeability in the vertical direction, as in Figure 5-4 from Reference 5-1. This figure shows that even when large horizontal permeabilities are found, the vertical permeability can be quite small.

The impermeability of strata at many natural sites suggests the possibility of tailoring the properties at a site to more closely fit those desired for siting an underground nuclear power plant. This possibility has not been examined in depth in this study but appears to be a viable option. It should be feasible, for example, to introduce a layer of clay containing material at the surface above the plant extending for several hundred yards. It may also prove desirable to introduce grout at some depth in the rock through drill holes from the surface or from the cavity during construction. Grouting of rock media is an established procedure in many situations during excavation and construction. It is recommended that further investigation of the effectiveness, cost, and procedures for tailoring a site be included in future investigations.

5.3 GROUNDWATER

The loss of contaminated water from the reactor chamber following an accident could in principle pose a hazard to public water supplies, and must be considered in site selection. Strontium-90 (half-life 28 years) and cesium-137 (33 yrs) are two of the principal isotopes of concern from this standpoint. The likelihood of this happening should be quite small if a drainage-sump system is introduced. Site-specific factors which would require consideration are proximity of aquifers, distance to wells or discharge area, "flow" rates, and mineral composition of the aquifer. The significance of the flow rates and distances to wells or discharge area is clear. The mineral composition is important because of the process of ion-exchange, which acts to retard the rate of movement of the radioisotopes relative to that of the groundwater, and to dilute the concentration at any given distance. The effect is to disperse and delay the arrival of the radioactive contamination front at any point downstream.

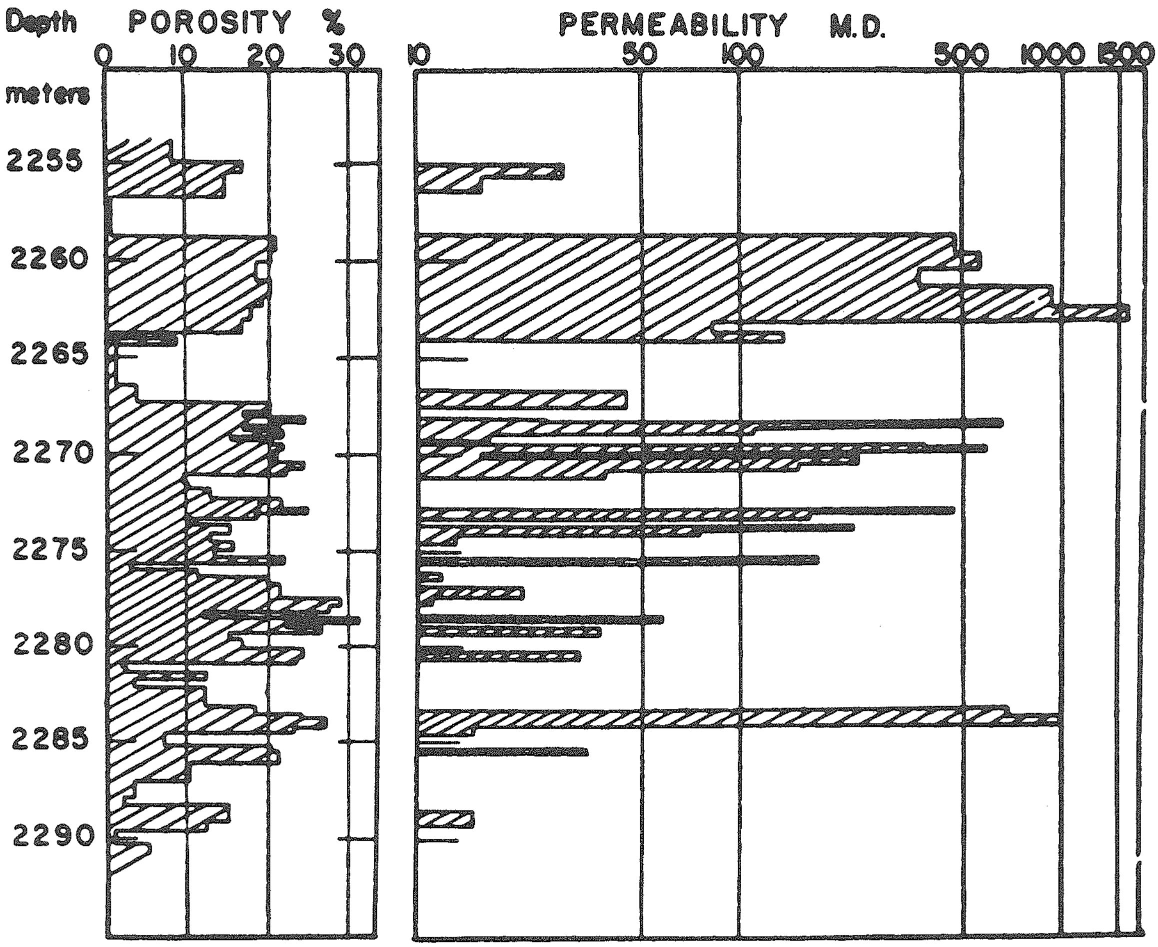


Figure 5-4. Section Through Springhill Sandstone (Chile)

Flow rates of ground water in aquifers are typically rather small, generally not greater than 5 ft per day, nor less than 5 ft per year. "Average" flow rates should be used with caution, however, because of the variability of permeability, as mentioned earlier. The relative proportion of radioisotope ions adsorbed in the solid phase of the aquifer minerals at equilibrium is given by the distribution coefficient K_d:

$$K_d = \frac{\text{ions in solid phase}}{\text{ions in solution}} \times \frac{\text{volume of solution}}{\text{weight of solid fraction}}$$

where "volume of solution" and "weight of solid fraction" refer to unit volume of aquifer material. The rate of transport of each radioactive species is reduced, relative to the groundwater flow rate, in proportion to its distribution coefficient:

$$F_i = \frac{F_w}{1 + \rho K_d}$$

where

F_i = rate of transport of ionic species i

F_w = flow rate of water

$\rho = \frac{\text{weight of solid fraction}}{\text{volume of water}}$ (unit volume of aquifier)

i. e., $\frac{\text{gms. of solid}}{\text{ml of water}}$

Some representative values of K_d are given in Table 5-2 (Ref. 5-3).

Table 5-2

REPRESENTATIVE DISTRIBUTION COEFFICIENT

Medium	Distribution Coefficients (ml/gm)	
	Cesium	Strontium
Hanford Soil	4500	420
Nevada Sandy Loan	4500	650
Savannah River Mixed Soils	90	90

As an example, assume an aquifer having the properties of Hanford Soil. The value of ρ is around 4 or 5 gms solid/ml of water for typical aquifers. Assume also a water flow rate of 1000 ft/yr. Then the average transport rate for cesium-137 is:

$$F \approx \frac{1000 \text{ ft/yr}}{20000} \quad \text{or } .05 \text{ ft/yr}$$

and for strontium-90 about 0.5 ft/yr.

5.4 FIELD MEASUREMENTS OF PERMEABILITY

For reasons discussed in Section 5.2, the permeability at a candidate site must be measured in situ. Reference 5-4 reports results of such measurements in granite, by the Swedish State Power Board, in pursuance of the Swedish nuclear siting program. For these tests, a horizontal hole was drilled to a length of 168 meters in the side of the Rävfjäll mountain. The last 50 meters of the bore were sealed off and pressurized with air. At the surface of the mountain, about 27 meters above the bore hole, 50 short holes (10 meters) were drilled at 45° to the vertical, so as to intersect fissures in the rock. These fissures were described as narrow, open and free of water. Air from these 50 bore holes was collected in glass containers filled with water, and volume seepage rates were determined as a function of chamber pressure. In other tests, krypton-85 was injected as a tracer and the transit time from the chamber to the surface was measured for different chamber pressures, in steady-state flow. It was found that the transit time of the tracer through ~ 25 meters of granite was 1.1 hours for a bore hole pressure of 3.3 atm, and ~ 2.8 hours at a bore pressure of 1.9 atm. Volumetric flow at the higher bore pressure was 3.3 m^3/hr at 1 atmosphere.

With plausible assumptions, it is possible to estimate the permeability of the Ravfjall granite from the data given. We assume that the flow is radial in the vicinity of the bore hole but ultimately is channeled through fissures to the surface. The seepage rate is about 900 cm^3/sec at 1 atmosphere, for 3.3 atm in the bore hole. Darcy's equation for steady-state isothermal flow in cylindrical symmetry is

$$\bar{Q} = \frac{2\pi Kh \ \Delta P}{\mu \ln\left(r_e/r_b\right)}$$

where
- μ = viscosity of air (centipoises)
- h = length of bore hole (cm)
- r_b = radius of bore hole (cm)
- r_e = drainage radius, at 1 atm (P_e)
- ΔP = $P_b - P_e$ in atm
- $\bar{Q}$ = flow in $\frac{cc}{sec}$ at $\bar{P} = (P_b + P_e)/2$
- K = permeability in darcies

The logarithmic term is insensitive and is in the range 5-7 numerically (r_e is not known and r_b is not given). Hence, the permeability is of the order

$$K = \frac{\bar{Q}\mu \ \ln(r_e/r_b)}{2\pi h \ \Delta P} = \frac{(900/2.1) \times (2 \times 10^{-2}) \times 7}{6.3 \times 5000 \times 2.3} \sim 10^{-3}$$

or 1 millidarcy. The assumed permeability in the idealized calculations earlier in this section was varied from 1 to 10 millidarcies. Because this flow takes place through narrow fissures (the bulk porosity of the rock is much smaller than for a sandstone) the rate of advance of the radioactive tracer through the rock is higher than it would be in a rock of similar permeability but higher porosity.

These measurements emphasize the importance of open joints and fissures and the consequent need for in situ testing of a candidate site. Although the tests just described are informative, they could not be regarded as adequate investigation of a site, but only as the initial phase of a site study. For instance, it was observed in these tests that as much as

30-40% of the leakage from the pressurized bore hole took place through a relatively confined area of faulted rock. One would want to see the effects of grouting of such areas. It would also be of interest to see the effects of cavity liners, surface soil layers, and so on.

SECTION 6

OPERATIONAL CONSIDERATIONS

6.1 NORMAL OPERATION

The operations normally associated with a nuclear power plant are those required when the plant is operating to produce power, and those conducted when the plant is shut down for reactor refueling, plant inspection, or repair. Some maintenance functions for a surface plant are accomplished while the plant is operating. Major maintenance is accomplished during periods of either routine or emergency shut-down and during annual reactor refueling. It is expected that an underground nuclear power plant could be operated in much the same manner. Access for routine maintenance to an underground plant will be the same as in comparable areas of the surface plants. However, less working space may be available in some areas, making the performance of some functions more difficult.

The difficulty and long time needed to remove large pieces of equipment to the surface from an underground plant suggests that within the technical resources of the plant all maintenance and repair be accomplished in place. In order to accomplish inspection and repair some equipment requires disassembly. Some pieces of equipment, such as feed water heaters and turbine cases, when disassembled occupy a great amount of additional floor space. Also the disassembled equipment pieces are very large and heavy, making compact storage difficult. The need for temporary storage space for these items can be provided by increasing the excavated chamber sizes at some expense in the cost of the chamber. Removal of the pieces to the surface may be a more attractive alternative. The permanent installation of hoisting machinery of much greater capacity than needed for normal maintenance would also be expensive unless the hoisting could be done with temporary equipment. The trade-off evaluation of such issues should be accomplished as details of the underground design are further developed.

The method of transporting radioactive material from the surface to the plant was briefly considered in the study. Handling of the spent fuel appears the most difficult. Safety considerations restrict the lifting height of containers for spent fuel to their corresponding impact strength. A long simple lift up a deep shaft would not be a satisfactory access for spent fuel capsules under present procedural guidelines. A tunnel, or possibly an inclined shaft, would be more satisfactory. This tunnel or inclined shaft would be greater in length and consequently more costly than a simple shaft but might also be used to provide heavy equipment access during excavation and construction of the plant. Other radioactive wastes in liquid and solid form could also be removed by truck through such a tunnel access to the plant. An alternate method of pumping the liquid waste to the surface appears less attractive because of special physical protection possibly needed for long pipe lengths carrying radioactive liquids.

Several unfavorable issues of underground plant operation and maintenance have been identified. Other factors unique to undergrounding are an asset. The underground plant receives natural protection from the elements such as rain, dust, sand and snow. For surface plants special treatment in the design of equipment is necessary to protect against elemental effects, largely eliminated in the underground plant. All weather shirt-sleeve working conditions are also a natural benefit of undergrounding. The importance of such benefits and the degree they offset the disadvantages are not completely known at this time.

6.2 CONSTRUCTION

It is assumed that the underground power plant will be sited near the coast. The costing exercises performed in Section 7 assume about 250 ft of cover over the plant and a location 2,000 ft inland from the water line. Figure 6-1 is an idealized layout of the power plant with a separation of the main galleries equal to twice the width of the galleries. A machinery assembly area is located at one end of the turbine generator

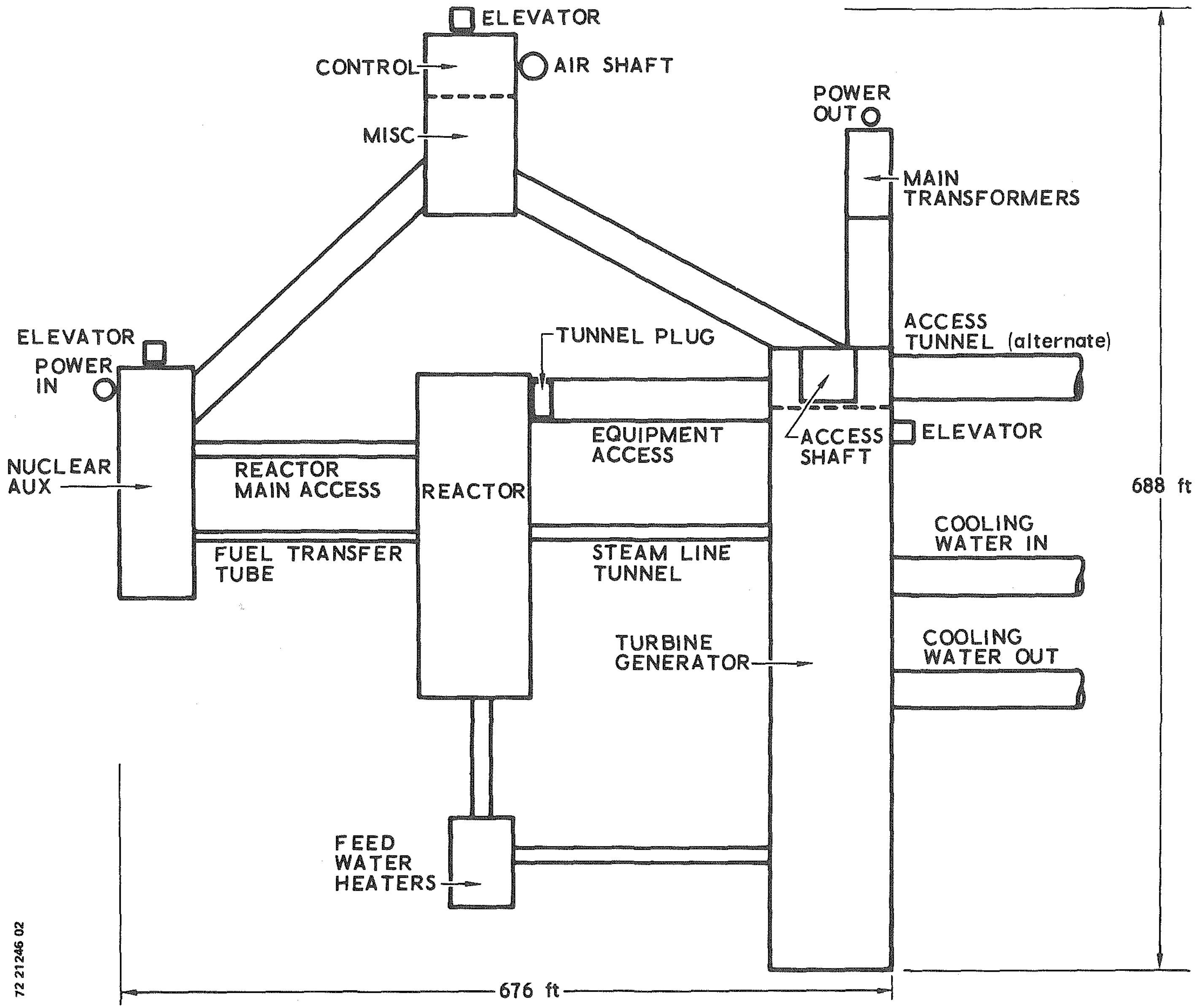


Figure 6-1. Sample Layout Underground PWR Nuclear Power Plant

gallery where major components could be lowered through a large access shaft or brought in through an access tunnel. These components will then be moved to the various galleries via a series of tunnels. It is assumed because of the very large dimensions of the galleries that excavation of the rock in the galleries would be accomplished by drill blast and muck method using benches starting at the spring line of the roof and cutting several benches to reach the floor level. The cooling water tunnels could be used for muck removal and the rock used for erosion control and construction of a break water at the shore line. Because the other tunnels are relatively short, it is assumed that they would be formed by drill blast and muck method. The cooling water tunnels could be constructed by using a boring machine or by conventional methods.

Depending upon the quality of the rock, it may be necessary to cast the roof and walls of the large galleries as the excavation is taking place for protection of the workers. It may be necessary to line the walls and roofs with rock-bolted wire mesh and gunite.

The excavation would be mined by the drill, blast and muck, bench method. A possible improvement in smooth wall excavation could be accomplished by wire sawing the long walls of a horseshoe room prior to benching. This would prevent damage to the adjacent rock during the blasting and may be less expensive than conventional smooth wall blasting. To accomplish this it would be necessary to use the bench method and, at the corners of a gallery, small pits would be excavated and the wire sawing equipment would be installed. The wire saws would then cut the rock the depth of one bench. The bench would then be removed by conventional means.

Excavation of the galleries would go on simultaneously with access to the galleries by interconnecting tunnels. Because of the large volume of rock in the galleries the use of a conveyor system is postulated to remove the rock through one of the cooling water tunnels. If the rock is of good quality, some of it could be stock piled for aggregate for the concrete required to line the chambers.

One way of accomplishing the fabrication of the liner is shown in Figure 6-2. The specially formed angles would allow all welding to be accomplished inside the chamber. The back angle would be welded to the liner. This would be followed by radiographic inspection of the welds. Next the front angle would be welded to the plates and radiographic inspection of those welds would be made. The angles form a relatively flexible joint to take up plate thermal expansion. An alternative would be to include a system of closely spaced plate anchors. Shoring would then be placed to hold the plate in place and concrete would be placed behind the plate.

The construction of the underground facility not including equipment installation might take about four years as shown in Figure 6-3.

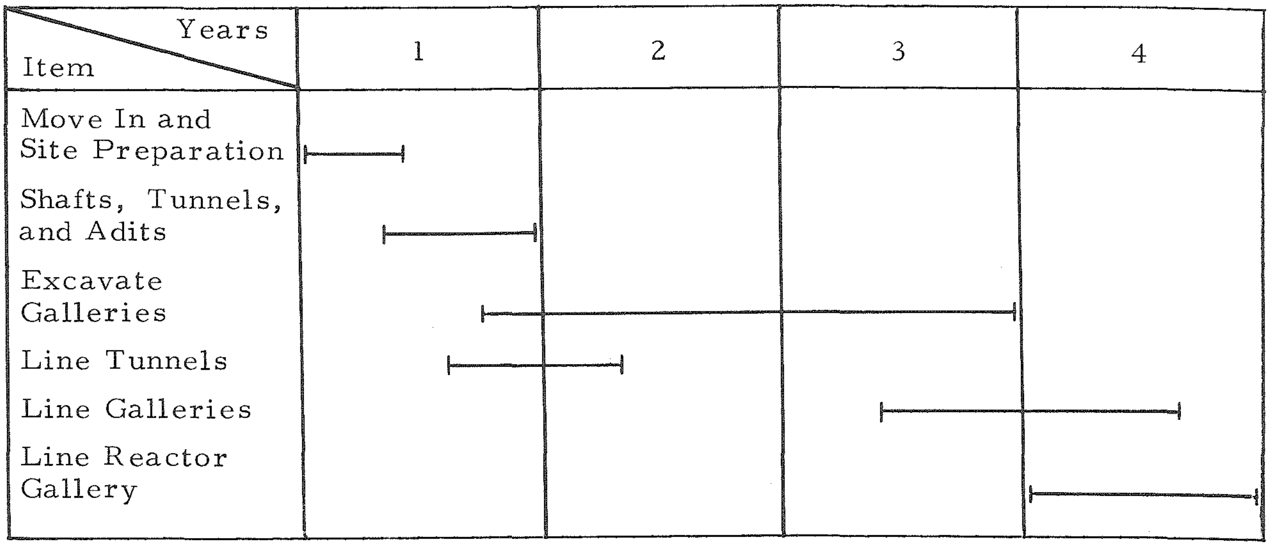


Figure 6-3: Postulated Construction Schedule

This is a conservative estimate based on an excavation rate of 1000 cubic yards per day in the main galleries after the roof arch has been formed. Rates of this magnitude have been achieved on large hydroelectric power plants. It should be noted that weather conditions should not hamper an underground construction job.

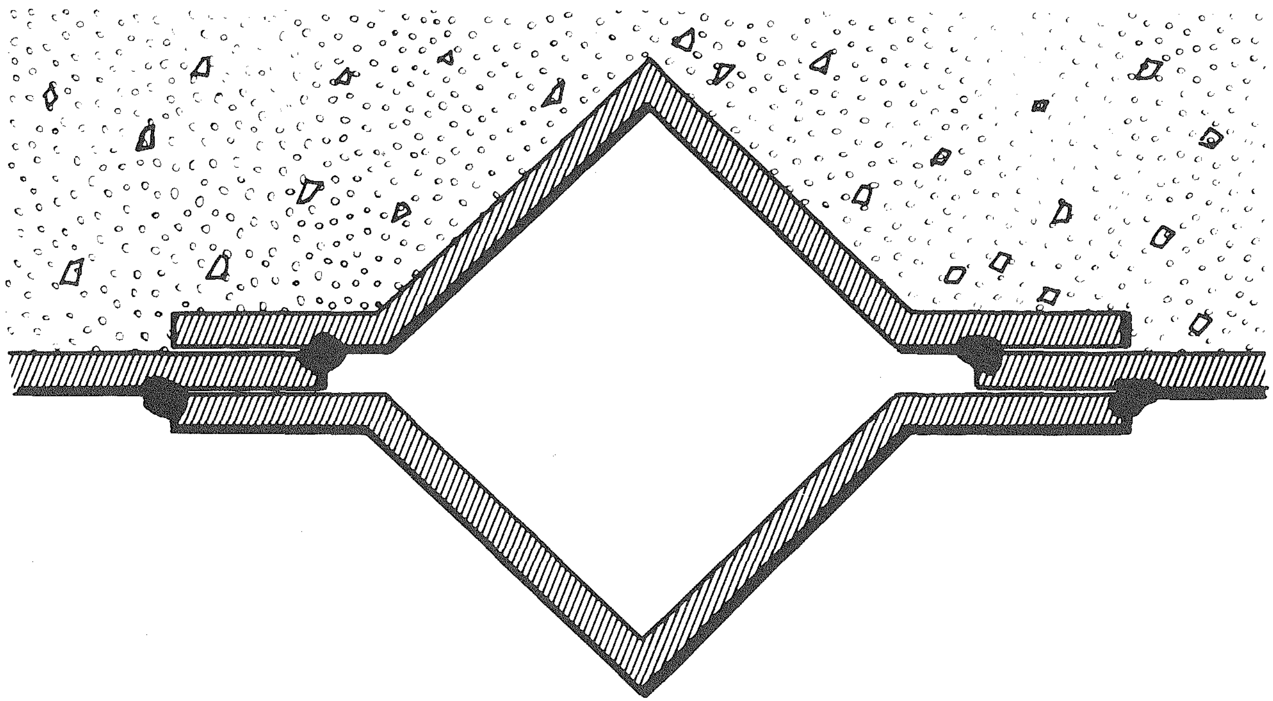


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Figure 6-2. Reactor Chamber Liner Joint Section

SECTION 7

COSTS

7.1 COSTING APPROACH

The method of costing the completed underground plant originally planned in this study was to obtain the cost breakdown for an existing surface plant of the same size and subtract the costs for the construction of the buildings and equipment installation. Then add to this cost the cost of construction of the underground galleries and tunnels together with the cost involved in the installation of the equipment. The installation costs will be higher because of the additional task of lowering the equipment through the access shafts and moving it horizontally through the tunnels to the various galleries. Some equipment costs will also vary with some higher and others lower. The critical item in this costing exercise is that of obtaining a comprehensive cost breakdown on an existing surface nuclear plant. At this time these costs, are not available; therefore, cost estimates have been made for the excavation and liners only. A more detailed examination of the cost differential for underground plants should be accomplished in future studies.

7.2 EXCAVATION

The baseline costing model is for a granite site having good quality rock with an RQD greater than 0.80 to 0.85. This assumes that the area contains homogeneous rock which is free from faults and has tight joints at a relatively large spacing. This type of rock would not require temporary supports during the excavation phase although it may require the use of some rock bolts and chain link fencing as a safety item to prevent injury to the workers. The baseline configuration of the galleries is a modified parabolic horseshoe design which minimizes stress concentration in the roofs and walls. The excavation costs were analyzed parametrically to include other materials such as sandstone and limestone of equal RQD value. Also considered are rocks of poorer quality where some shoring would be required during the excavation phase. A plant of this size would not be constructed in poor quality rock because of high construction costs and the difficulty of the construction process.

The unit costs for the baseline configurations are based on the following material properties:

seismic force	0.5 g containment areas
	0.25 g all other areas
rock quality	80-85 RQD
rock type	granite
steel yield strength	60,000 psi
concrete compressure strength	4,000 psi
reinforcing steel	1%
arch rise to span ratio	1/8

The unit cost derived from averaging several large underground hydroelectric power plants and the Cheyenne Mountain NORAD Facility costs are:

shaft excavation	\$60 per yd^3
tunnel excavation	\$20 per yd^3
gallery excavation	\$20 per yd^3
reinforced concrete	\$100 per yd^3 inplace
steel reactor liner	\$20 per ft^2

These costs are affected by many factors and are thought to be conservative. Harza Engineering Company, Chicago, Illinois has generally used lower unit costs in estimating large excavations.

Franklyn C. Rogers of the Harza Company, states in the October 1971 Bulletin of the Atomic Scientists:

> "Assuming good rock conditions, reasonably straight-forward excavation progress, no more than occasional supporting of the rock and a normal level of precautionary measures, it is entirely feasible to achieve an excavation cost in the neighborhood of \$5 to \$10 per cubic yard. If local practices demand more than usual safety precautions and other restrictive limitations which prevail in certain urban areas, the excavation cost would probably be in the neighborhood of \$15 per cubic yard. The figures would be

> used for pricing of underground chambers to house reactors and, if such were placed underground, turbines, condensers and generators."

It is possible that this estimate is based on sandstone or limestone such as encountered in the Chicago Sewer Project. The cost of excavation will vary with the type of rock to some degree. Generally speaking, excavation in sandstone or limestone should be 15% to 20% less than granite. Therefore, in this study we have used a figure of $17 per cubic yard for this type of rock with an RQD of about 85.

If the RQD is around 60, it is assumed that woven wire mesh and rock bolts will be required during the excavation phase to stabilize the roof and walls. This would add a cost of $2 per square foot of surface area.

If the site contains poor rock with an RQD near 40, more extensive work would be required to support the walls during excavation. The use of wire mesh, rock bolts, roof and wall ties and shotcrete would cost approximately $6 per square foot of surface area.

In these cases of poorer rock, it is questionable if openings of the size required could be made unless incremental construction of the roof and wall liner is accomplished during the excavation phase. This would require a longer construction period and also tend to increase costs.

7.3 LINER COSTS

The horseshoe configuration for the galleries was selected to reduce the thickness and cost of the liner. Preliminary designs were prepared for several configurations. A flat wall and arch configuration with a seismic loading of 0.5g required a wall that was twice as expensive as that required for the horseshoe configuration. In the case of a 1g loading the wall was three times as expensive. One component of the wall which has a fixed cost is the steel liner required for the reactor gallery. This liner was assumed to consist of a 3/8 inch steel plate with steel angles covering both sides of all joints to minimize leakage.

The volume between the angle and the steel plate could be pressurized with a gas so that there will be no leakage from the reactor gallery into the concrete walls and surrounding rock through the welded joints.

The unit costs used for the liners are $100 per cubic yard for concrete in place using 1% reinforcing steel. The steel liner for the reactor chamber was costed at $20 per square foot which includes radiographic inspection of all welds and pressurization of the welded areas.

7.4 SUMMARY

Costs of three configurations have been developed as a baseline. The costing was performed by developing preliminary wall designs for each of the items and computing excavation volumes, reinforced concrete volumes and cost of a steel liner in the reactor chamber where applicable. The baseline construction costs are for excavation in a granitic rock with an RQD of 85 or greater with costs of $17.7 million for the BWR minimum modification configuration, $15.5 million for the reconfigured BWR, and $13.7 million for the PWR configuration.

If the power plants were sited in sandstone or limestone instead of granite, the excavation costs would be $17 per cubic yard instead of $20 per cubic yard. This would result in a 15% savings in excavation costs or a savings of $1.8 million for the BWR minimum modification configuration, $1.6 million for the reconfigured BWR and $1.4 million for the PWR configuration. A savings of 15% in excavation costs would result in about a 10% savings in the overall construction costs.

The effects of rock quality on the excavation costs are more severe because of the requirements for temporary shoring, rock bolting, wire mesh, guniting, etc., to keep the rock in place prior to construction of the reinforced concrete liners. In addition, the liners must be thicker to support the rock. These changes reflect an increase in the underground construction as shown below. For an RQD of 60 the cost for the BWR minimum modification configuration is \$33.4 million or an increase of 89% over the baseline costs, for the reconfigured BWR the costs are \$26.4 million, an increase of 70% and for the PWR the costs are \$23.6 million representing an increase of 72% as shown in Figure 7-1.

In the case of a poor rock with an RQD of 40 the cost for the BWR minimum modification configuration is \$52.1 million which is an increase of 194% over the baseline costs; for the reconfigured BWR, costs are \$40.0 million an increase of 158% and for the PWR, \$34.7 million or 153% increase. These large increases are mostly due to walls up to 25 feet thick. Obviously it is impractical to construct an underground facility such as this at a site characterized by significant seismic loads (i. e., non negligible) and poor rock quality (Figure 7-2). The cost impact of reduced rock quality (RQD) indicated in Figure 7-2 is, of course, only broadly representative with large variations likely at actual sites.

Seismic acceleration levels effect the thickness of the reinforced concrete cavity liners in a linear manner. Thus the effect on costs are also linear as shown in Figure 7-3.

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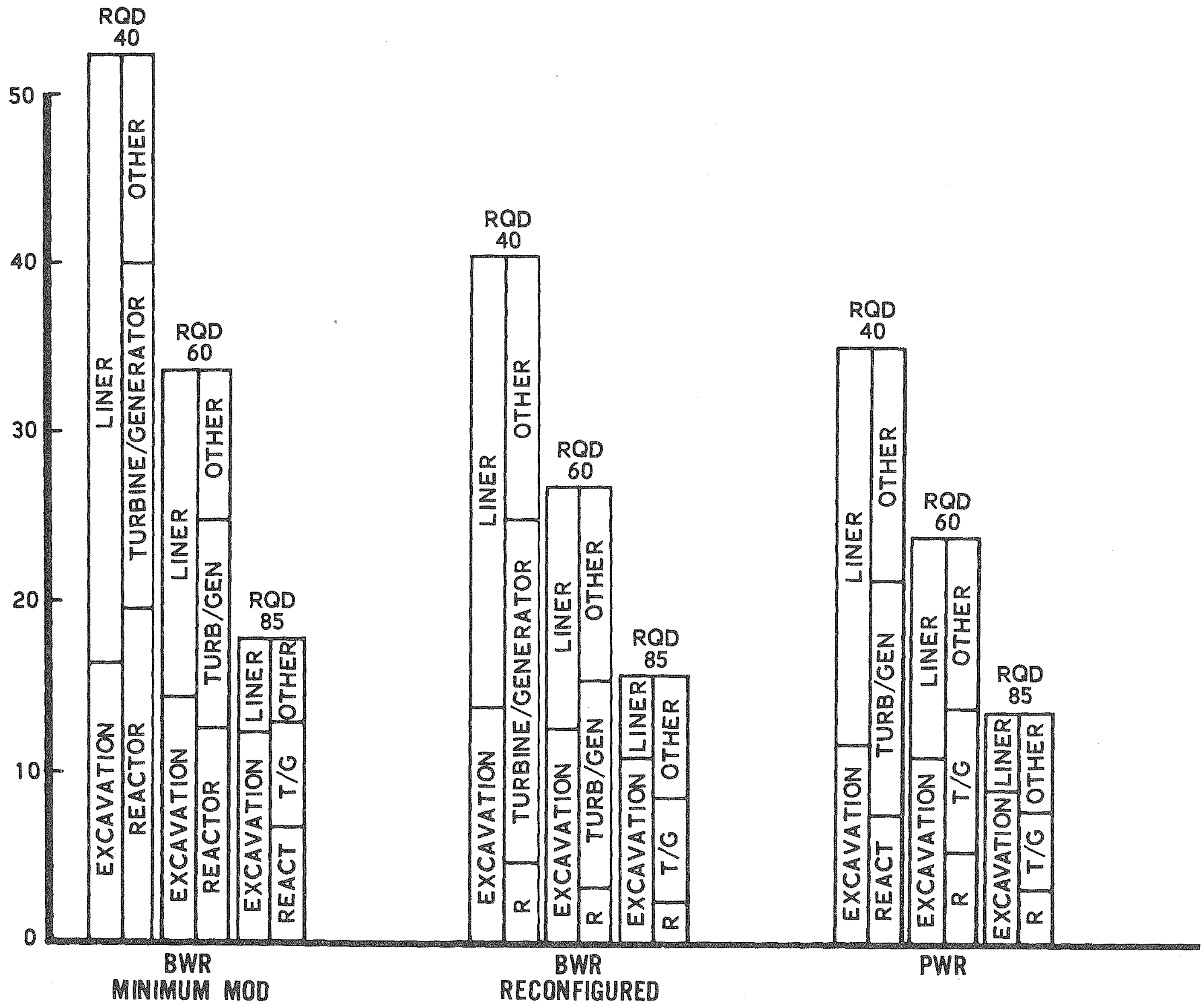

Figure 7-1. Gallery Construction Cost vs RQD

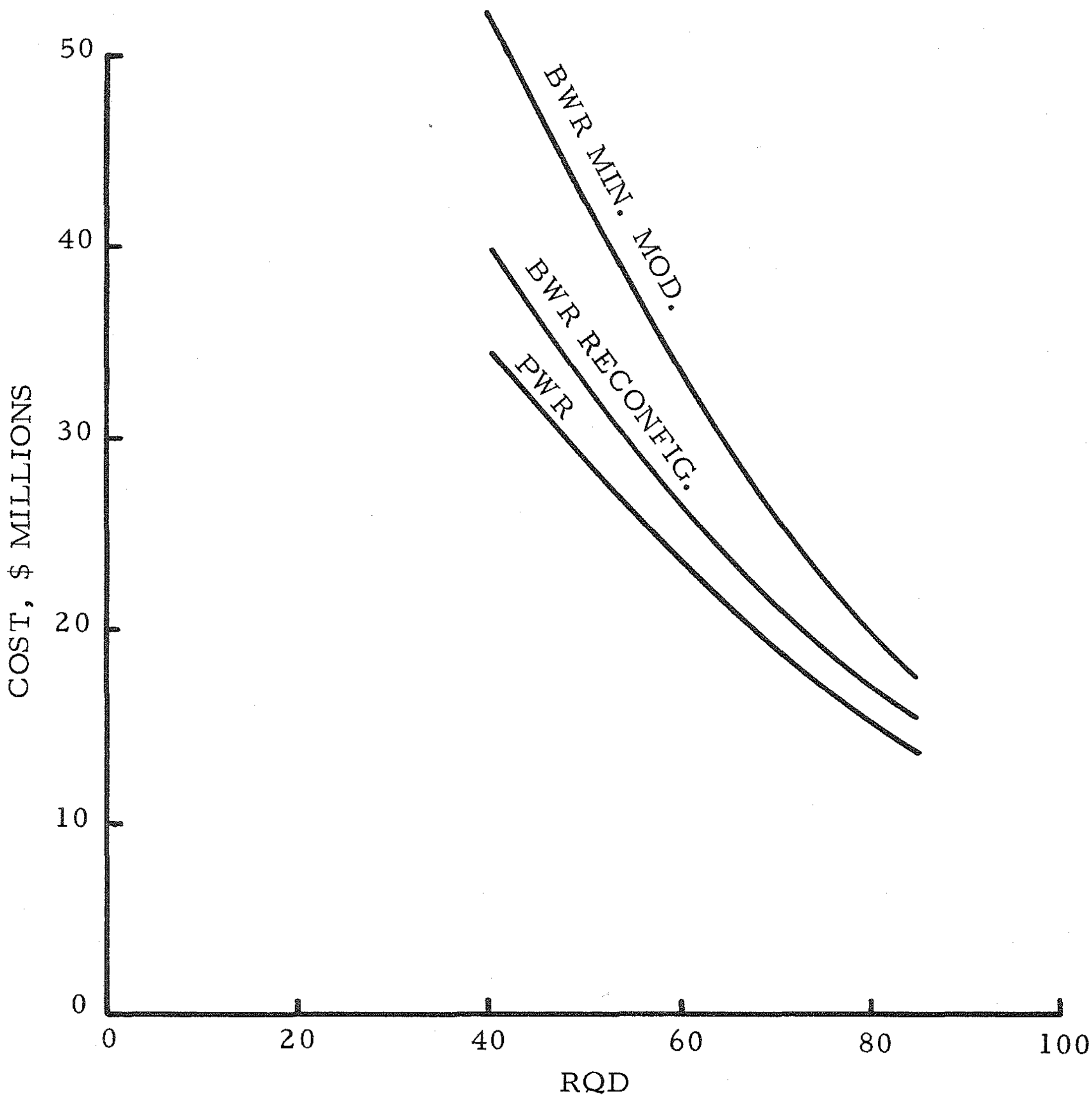

Figure 7-2. Gallery Construction Cost vs RQD

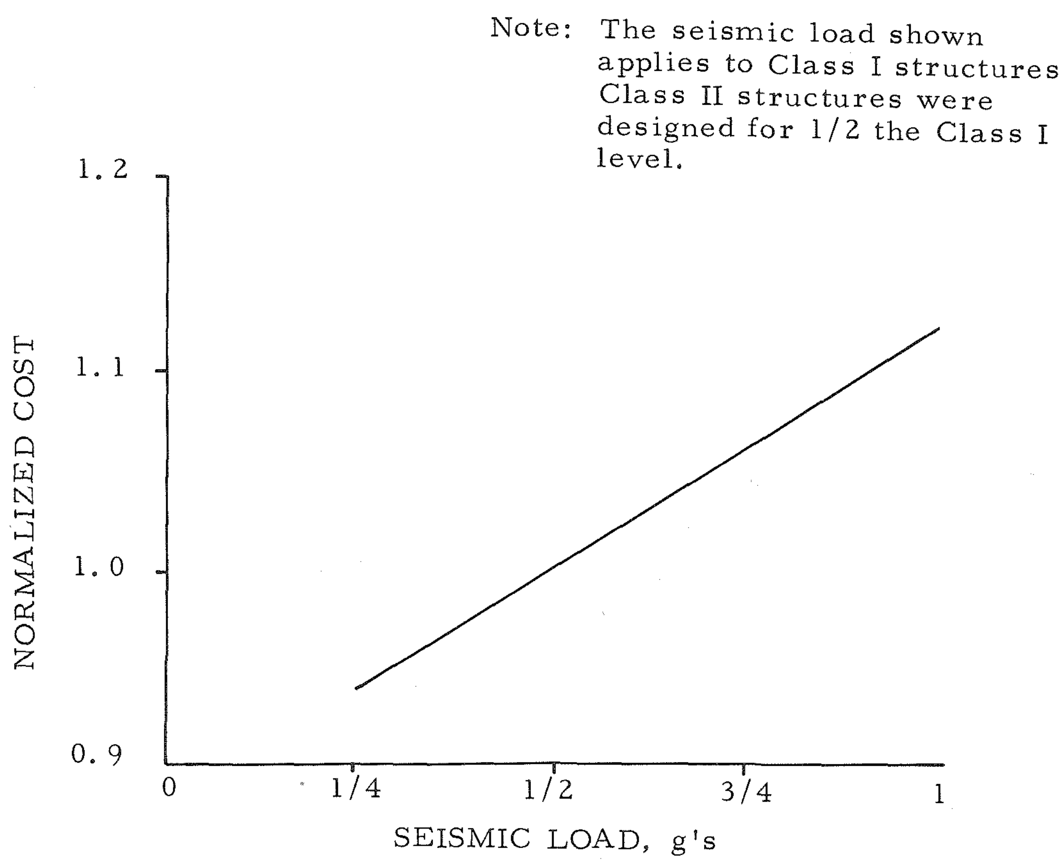

Figure 7-3. Gallery Construction Cost Sensitivity to Seismic Load

SECTION 8

SITING CRITERIA

In the process of completing the study activities summarized in previous sections, information has been derived relative to the characteristics of the site where an underground nuclear power plant can be sited. While it has not been possible to develop rigid acceptance criteria, summary statements can be made relative to preferred site characteristics. These characteristics have been reviewed in an effort to identify those factors that are mandatory and those that are highly desirable. They are further subdivided by the requirement that dictates the site survey guideline. For example, the permeability and porosity of the rock are dictated most directly by the requirements for containment. If a plant is to be sited underground primarily to improve containment and reduce the population center separation distance, then the range of acceptable permeabilities and porosities will be more demanding than if other objectives are sought. Permeability and porosity are of indirect importance to the constructability of the plant. Site selection criteria for underground nuclear power plants will remain somewhat uncertain until the objectives of such siting is determined.

In formulating siting criteria, it is tempting to identify the most restrictive and critical parameters to minimize the number of sites that must be evaluated in detail. In principle, one might imagine a comprehensive set of detailed contour maps for each of the critical siting parameters. These contour maps could be transferred to a transparency with unacceptable areas made opaque. By superimposing a set of these transparencies, one for each criterion, and holding them up to the light, only a few points of light will be seen corresponding to acceptable sites. Unfortunately, such contour maps do not exist. Even if such maps were available, it is possible that no sites would be found that meet all criteria. This does not rule out the possibility of underground siting. Site surveys should adopt broad guidelines to assure that the best siting areas are identified and recognize that final site selection may involve some compromise away from the ideal site.

The construction of site criteria should also be influenced by a judgment relative to what is available and how the information might be used. Perhaps the most obvious example of the use of available data is the specification of igneous or sedimentary rocks. The geologic origin of the rock is unimportant in itself, but such data is readily available and meaningful inferences can be drawn from the known characteristics of these classes of rocks. Conversely comprehensive permeability maps do not exist and the cost of developing such maps would be enormous. The formulation of site selection criteria must be influenced by the type of information that is available and the procedures that can be practically used to locate sites.

The broad siting guidelines identified in this study are summarized in tabular format below.

Requirement	Guideline
Excavation Safety, Costs, and Structure Loads	RQD The RQD index has been used as a coarse index of rock quality. Large span requirements suggest an RQD greater than 75% corresponding to good and excellent rocks is strongly desired. Sites with an RQD less than 60% will lead to very costly excavations and are therefore unacceptable. Intermediate RQDs are marginally acceptable. Rock Strength & Modulus No numerical criteria are specified for rock strength or modulus because most rock media

that comply with the RQD criterion will inherently possess sufficient strength. (Some very weak sandstone or shale may have inadequate strength but high RQD). These parameters should be recorded as part of siting surveys.

Geometric

Area

The configurations developed in this study require an area of at least 700 x 700 feet. This size is judged a mandatory minimum. An area of at least 1000 x 1000 is a reasonable "desired" criterion.

Depth

The large galleries postulated in this study require a thickness of mechanically uniform, competent rock of 180 feet and preferably 210 feet. This thickness should be topped by at least 60 feet of additional rock, weathered rock or soil cover such that the base of the construction region is at a depth of at least 240 feet.

Ocean Proximity

The present study has assumed the use of ocean water for cooling. Economics will dictate the minimum acceptable distance from the shore. A reasonable maximum of five miles might be used in early site surveys as desirable with no limit specified as mandatory.

Topography

Surface topography is largely immaterial for selection of the underground plant. An average slope from the ocean shore should be large enough to meet the depth of cover requirements consistent with economic lengths of cooling water tunnels. A target rise of one in five from sea level is desired.

Geologic

Rock Type

No criterion is specified limiting site selection to igneous, sedimentary, or metamorphic rocks. Thick sedimentary deposits may be favored for containment. Uniformity of the mechanical properties is judged more important than geologic classification.

Tectonic Stresses

Unusual tectonic stresses should be avoided particularly where significant horizontal stresses are encountered.

Containment

Porosity and Permeability

Sites should not be excluded at this time solely on the basis of permeability or porosity. These quantities should be recorded during site surveys. Low permeabilities are desired, preferably below 10 millidarcies. High porosities are also desired but no numerical limit is proposed.

Saturation

Saturation of the pores is judged a favorable characteristic. Natural flowing aquifers should be avoided.

Jointing

Jointing is an unfavorable characteristic but cannot be avoided. Containment will be improved where joints are well cemented preferably with clay containing materials to impede fluid flow and reduce fracture permeability.

Seismic

Fault Proximity

A quantitative fault separation distance has not been established. Faults should not run through the site or immediate area.

Discontinuities

Major mechanical discontinuities should be avoided even though not associated with faulting, e.g. a sedimentary-igneous boundary should not cross the site volume.

Other

Criteria for population centers, electrical system compatibility, political exclusion (e.g. Indian Reservations), etc. have not been formulated.

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APPENDIX I

EUROPEAN UNDERGROUND NUCLEAR PLANTS

Several underground nuclear power plants have been constructed and successfully operated in Europe. A summary of the significant features of each underground plant is presented in Table I-1. An obvious feature of all the plants is that they are relatively small in size (8.5 to 266 MW_e). The motivation for undergrounding the plant appears to be insurance of containment of accidently released radioactivity and also physical protection from damage due to hostile military action. The sites are all in rock and the depths and covers are nominally between 15 to 30 meters depending on the site. The plant at Chooz, France has the largest span of 69 feet. In all but one of the plants, the turbine generator is located at the surface. One of the plants (Agesta) supplies steam for space heating of a nearby large apartment complex. Since the construction of the Halden Reactor in 1955, Norway has maintained a growing interest in underground nuclear plants. Recently some details of their effort were disclosed at the 1971 International Conference on the Peaceful Uses of Atomic Energy. Norway's effort included study of plant configurations of 500 MW_e size for BWR and Advanced Gas Cooled Reactor (AGR) with siting at the surface and underground. The design approach was to place the entire plant underground with the reactor and turbine generator in a common gallery. The span of the largest gallery was 30 m (98 feet) and total excavation 264,000 m^3 (345,000 yd^3) or about 19 cu. ft./Kw.

TABLE I-1

UNDERGROUND NUCLEAR POWER PLANTS

Name and Location	Size	Purpose	Configuration		Depth	Reactor Chamber Dimensions (feet)
			Turbine Generator	Reactor		
Halden Norway (BHWR)	25 Mwt	Experimental	None	Rock Cavern	98 feet	98' long 85' high 33' wide
Agesta Stockholm, Sweden (PHWR)	80 Mwt/ 20 Mwe	Heat & Power	Above ground at reactor grade level	Rock Cavern	49 feet	88' long 66' high 54' wide
Chooz Ardennes, France	266 Mwe	Power	Above ground	Rock Cavern	--	138' long 146' high 69' wide
Lucerne Switzerland	30 Mwt/ 8.5 Mwe	Experimental/ Power	Rock Cavern	Rock Cavern	--	--

APPENDIX II

FACTORS AFFECTING THE SIZE OF UNDERGROUND OPENINGS

by

Warren Pfefferle

Mining engineers have long associated increased size of excavation with increased costs, increased difficulty in construction, and a probable overall reduction in stability of the resulting excavation. While this opinion is nearly universally held by all those associated with tunneling, mining, and other underground excavation construction, it has never been quantified in any reliable fashion. Indeed, from a study of elasticity theory one would judge that any investigation into the effects of size on strength must go beyond the theories of elasticity and considerations of pure size itself. More than likely the observed size effects will be dependent upon the absolute strength of the rock medium, variations in strength within the rock mass, and the exact nature of any nonuniformity or anisotropy. However, the major factor in the observed reduced strength is probably the occurrence of discrete weaknesses in the rock, that is, presence of joints, faults, and microfractures. As the sample of rock increases, the probability of finding a worst case weakening factor increases. Based on this one might expect that a long, small diameter tunnel with a surface area equal to a very large cylindrical chamber would have the same strength. Experience would probably not bear out this conclusion because still another factor, which is more social than engineering, enters into the description of tunnel rock excavation failures -- namely, does failure stop the construction for any appreciable period of time, say greater than a shift, or does the failure result in a hazard to personnel or loss of life? Assume a given defect occurs under two circumstances, first near an eight-foot diameter tunnel, and second near an eighty-foot radius arch. In the first case a casual arrangement of steel sets and lagging would probably prevent the falling rock from causing loss of life. When the weakened tunnel section is observed, the shift foreman might scale the loose rock and make repairs with materials and manpower available during the ordinary work shift. In the second case, however, the same initial failure would propagate on a large scale and involve such a large area that a major calamity would ensue. Thus the consequences of a given rock defect or initial failure may vary nearly as the cube of the cavity size and, therefore, willingness to open a cavity might decrease something like the cube of the cavity size. Human factors, therefore, are likely to have a major impact in the final determination of just how large a cavity we desire for a given purpose.

Since limitations on cavity size are primarily based on experience rather than on a theoretical foundation, it is quite natural to examine existing cavities and how they have performed over the years. Several spectacular natural rock spans are familiar to most of us. First is Carlsbad Caverns made up mostly of Pennsylvanian and Permian limestones. The largest chamber in the Carlsbad Caverns is some 4,000 feet long, nearly 625 feet wide and 350 feet high. This cavity has been standing for thousands of years without benefit of roof bolts, steel sets, gunite or concrete. Any cementing or grouting of rock joints was concurrent with the formation of the cavity. Closer to Southern California area is the Rainbow Natural Bridge in southern Utah. This natural bridge arches 305 feet above the stream which meanders through the gorge beneath. It has a clear span of 270 feet--again, much larger than man-made excavations. Numerous similar natural bridges and sea caves are found along the coast of California where the soft sandstones and mudstones have been undercut by wave action. Two natural cavities in Virginia are also worth noting, first the Natural Tunnel in Scott County, Virginia, which is used as a railway tunnel by the Southern Railway. This natural excavation has a length of 900 feet averaging 75 feet in height with a clear span of about 130 feet. Natural Bridge, southwest of Lexington, Virginia, is in use as a highway bridge. It is a rock span about 90 feet long varying in width from 50 to 150 feet with a rock thickness of about 50 feet at the crown. It arches 200 feet above the narrow gorge of Cedar Creek. The foregoing natural rock cavities help place the excavations of man in their proper perspective. For example, Churchill Falls Hydroelectric Project, the largest of its type in the western world, has a clear span of about 81 feet in its largest chamber, the power house. This excavation is 972 feet long and rises to a maximum height of 154 feet. The rock material is designated as a homogeneous, granite gneiss. This rock is of high quality, and only moderate stabilization by such methods as rock bolting was required. For the roof of the power house rock bolt length varied up to about 45 feet. Another large successful underground excavation is the machine hall on Washington's Pend Oreille River. The turbine room is 477 feet long with a 76-foot span and a roof height up to 190 feet. Again, with minor exceptions rock bolts were used to support all excavations with 15-foot bolts on a 6-foot pattern used in the roof of the power house. Spot bolts up to 30 feet long were added where adverse jointing was encountered. Sprayed concrete and chain link fencing serve to contain rock falls and a weather roof of corrugated metal protects the floor below from water dripping off the roof. A somewhat larger underground power house was constructed in what was considered a bad rock condition at British Columbia's Portage Mountain Dam. The rock at this location consists of thickly bedded, medium grain sandstone and fine grain, grey to black siltstone and shale with some finely sheared zones and coal seams. The strike of the beds is parallel to the power house chamber and dips about six degrees downstream, resulting in a stair step or shingle pattern in the arched roof of the excavation. First, the roof was rock-bolted by bolts most of which extend over 10 to 20 feet to anchor in a massive sandstone stratum above the arch crown. Then a concrete arch roof, two and one-half feet thick at the center and four and one-half feet thick at the abutments provide a permanent lining in the cavity. This combination is considered to have been effective in stabilizing the 85-1/2 foot span by 890-foot long main excavation even though significant movements of the cavern roof were observed during construction.

Practical limitations on cavity size must consider the value of a large, clear span for a given project, including such imponderables as the potential loss of human life during construction, and unfavorable public reaction to additional unplanned expenditures to repair rock failures after construction has begun. A first approximation could be made on the basis of elasticity theory if the excavation is for a flat roof cavity in massive sedimentary rock. For example, in a massive sandstone or limestone having a flexural strength of approximately 400 to 600 psi, a clear span of about 100 feet would require a layer thickness of at least 40 feet to provide an adequate margin of safety. This layer thickness may either be a naturally occurring situation or a thickness that is built up by grouting and rock bolting the rock mass prior to opening the excavation to its full span. Similarly, though in perhaps a less clearly defined manner, a jointed rock medium could be studied to determine the largest and most significant defect likely to occur in the final excavation and then to assess the required degree of rock bolting or other rock stabilization required to retain the volume of rock likely to be involved in an incipient failure. In this latter case of a severely jointed igneous rock it is more than likely the design would have to proceed in step with actual excavation since it is unlikely that any program of exploration prior to the start of on-site construction would be thorough enough to provide the detailed jointing information that would be necessary for design purposes on a conservative and yet economical basis.

Turning now to the specific problem of underground nuclear power plants, we find that the desired clear span is relatively large with respect to the size of excavations that have been constructed for other civil works projects. There is also a need for substantial economy and a very high factor of safety in the finished construction. Both these factors plus the natural geology of the California coast make it a strong probability that such a power plant would be constructed in a sedimentary rock at a relatively shallow depth, say about 300 feet. Under these conditions a vertical ellipsoid or large diameter cylinder with a dome top would probably lend itself quite well to specialized excavation methods that would produce a large span cavity on a very economical basis. For planning purposes it is probably reasonable to consider dome diameters in the 120 to 200-foot range. Flat roofs might also be considered if detailed site investigation indicates exceptionally massive bedding of rock with a flexural strength in excess of about 400 psi.

TABLE II-1

LARGE ROCK SPANS

Name	Size	Rock Type	Date	Remarks
Natural				
Rainbow Natural Bridge Southern Utah	270' Wide 305' High Arch	Sandstone		
Carlsbad Caverns Carlsbad, New Mexico	625' Wide 300' High 4000' Long Irregular Shape	Limestone		
Mammoth Dome Mammoth Cave, Kentucky	150' Wide 80-250' High 400' Long	Limestone		
Natural Tunnel Virginia	130' Wide 75' High 900' Long		In Use as a Rail Tunnel	
Natural Bridge Virginia	90' Span (50' Thick) 200' High 50-150' Wide	Limestone	In Use as a Highway Bridge	
Man Made				
Straight Creek Tunnel	42 x 44 (45 x 55 exc) 1873' Guage Zone 8940 Overall	Gauge Zone Granite	Current (1969-72)	
Liverpool Road Tunnel England	34' Ø 7000' Long	SS, More	1968	
Lucerne Highway Tunnel	34-1/2' Ø 2-5100'	LS, More		Sprayed Concrete Liner
Churchill Falls Labrador Hydropower Plant	81'Wide 154' High 972 Long	Granite/ Gneiss	1971	Largest hydroelectric project in Western world. 1000' rock cover ~$2\text{-}1/2 \times 10^6$ CY rock excavation
Morrow Point Colorado Hydro Power Plant	57' Wide 134' High 206' Long (400' Rock Cover)	Schist, Quartzite	1967	
Portage Dam British Columbia Hydro Power Plant	85' Wide ~55' High 890' Long	Sandstone, Shale, Siltstone	1970	Concrete roof arch, rock walls
Boundary Dam Washington	76' Wide 190' High 477' Long	Dolomitic	1967	Rock bolted wall and arch
NORAD Facility Colorado Springs	Multiple Chambers with 35 & 45' Clear Spans	Granite	1964	Rock bolt and mesh

APPENDIX III

DEPTH OF BURIAL ANALYSIS EQUATIONS

The equations used in the depth of burial analysis discussed in Section 4.1 are listed below. These equations are standard solutions for simple elastic problems and are derived and discussed in many texts. See, for example, "Rock Mechanics and the Design of Structures in Rock," by L. Obert and W. Duvall, John Wiley & Sons, Inc. 1967.

Spherical Cavity

Stress due to lithostatic load:

$$\sigma_{\theta_\ell} = \frac{-\gamma Z}{14-10\nu}\left(\frac{(9-15\nu)A^3}{R^3} - \frac{12A^5}{R^5}\right) + \left(\frac{\nu}{1-\nu}\right)\left(\frac{-\gamma Z}{14-10\nu}\right)$$

$$\left[\left(14-10\nu - \frac{(5-10\nu)A^3}{R^3} + \frac{21A^5}{R^5}\right) + \left(\frac{(9-15\nu)A^3}{R^3} - \frac{12A^5}{R^5}\right)\right]$$

$$+ \left(\frac{\nu}{1-\nu}\right)\left(\frac{-\gamma Z}{14-10\nu}\right)\left[\left(\frac{(30\nu-15)A^3}{R^3} + \frac{15A^5}{R^5}\right) + \left(\frac{(9-15\nu)A^3}{R^3} - \frac{12A^5}{R^5}\right)\right]$$

Stress due to cavity pressure:

$$\sigma_{\theta_P} = \left(\frac{1}{2-4\nu} - \frac{\nu}{1-2\nu}\right)\frac{A^3}{R^3}\,P$$

Cylindrical Cavity

Stress due to lithostatic load:

$$\sigma_{\theta_\ell} = -1/2 \left(1 + \frac{\nu}{1-\nu}\right) \gamma Z \left(1 + \frac{A^2}{R^2}\right) - 1/2 \left(\frac{\nu}{1-\nu} - 1\right) \gamma Z \left(1 + \frac{3A^4}{R^4}\right)$$

Stress due to cavity pressure:

$$\sigma_{\theta_P} = \frac{A^2}{R^2} P$$

Where:

- ν, Poisson's ratio
- γ, rock density
- Z, distance measured from ground surface
- R, distance measured from cavity center
- A, cavity radius
- P, cavity pressure

APPENDIX IV

FLUID FLOW THROUGH PERMEABLE MEDIA

The movement of liquids and gases through permeable media is governed by Darcy's law, which in its simplest form is

$$v = \frac{K}{\mu} \Delta P$$

where

v is volume rate of flow of fluid per unit area,
K is permeability of the medium,
μ is viscosity of the fluid, and
ΔP is pressure gradient.

By reason of the enormous internal-surface area of porous media, the flow is dominated by viscous resistance, and is therefore independent of inertial forces. At very high rates of flow (turbulent flow) deviations from Darcy's law set in, corresponding to an apparent decrease in permeability, i.e., Darcy's law overestimates the flow rate in this case. For our purposes, the law is valid or would conservatively overestimate flow rates.

The permeability of a rock sample is in principle a geometric property. Fluid properties enter the Darcy equation only through the viscosity, μ. Permeabilities are measured in the laboratory by driving dry air through small oven-dried specimens, and measuring the steady-state rate of flow at a fixed pressure differential across the sample. The permeability, K, is found from the equation

$$K = \frac{2\, P_o Q_o \mu L}{\left(P_i^2 - P_o^2\right) A} \qquad (1)$$

where Q_o is the volume rate of flow in cm^3/sec at pressure P_o

P_o = outlet pressure,

P_i = inlet pressure,

μ = viscosity of air, and

L, A = length, cross-sectional area of sample.

If pressures are measured in atmospheres, length in centimeters, and viscosity in centipoises ($\mu \approx 1$ cp for water, $\approx .02$ cp for air, $\approx .025$ cp for krypton and xenon at $\approx 300^oK$), then K is in darcies. For example, suppose the test core is 1 cm in length, 1 cm^2 in cross-section, P_i is 2 atm, P_o is 1 atm, K = 0.1 millidarcies, then

$$Q = \frac{10^{-4}}{2x10^{-2}} \; \frac{3}{2} = .0075 \; \frac{cm^3}{sec} \quad (1 \text{ atm}). \tag{2}$$

Permeabilities may also be measured in situ by pressurization of drill holes; the permeabilities in the vicinity of the Hard Hat nuclear test were determined in this way. (See Appendix V for a description of Hard Hat.) The value of K in the example calculation above is typical of the unshocked, in situ Hard Hat granite. An alternative form of Darcy's law for linear isothermal gas flow follows from continuity of mass:

$$\rho Q = \text{const} = \rho_o Q_o \tag{3}$$

and Boyle's law:

$$\frac{\rho_o}{Po} = \frac{\rho}{P} = \text{const} \tag{4}$$

or

$$P_o Q_o = P Q = \overline{P}\,\overline{Q}$$

where $\bar{Q}$ is flow at mean pressure, $\bar{P} = \frac{P_i + P_o}{2}$. Then

$$K = \frac{\bar{Q}\,\mu L}{\left(P_i - P_o\right) A} \quad \text{or}$$

$$\bar{Q} = \frac{K\left(P_i - P_o\right) A}{\mu L} \tag{5}$$

The spherical analyses discussed in Section 5.1 can be concisely summarized by use of the linear version of Darcy's law, Equation 5 above. Let D_p be the distance that gases might penetrate into the rock if the cavity pressure is maintained for a duration, τ. Also assume the following parameters:

f	=	porosity	=	.20 ~ "sandstone"
			=	.01 ~ "granite"
K	=	permeability	=	10 mdarcy (both)
τ	=	transient duration	=	1/3 hour

It is assumed that the rock porosity is available for gas flow, i.e., is not occupied by water. One can regard this model in the following way: Initially we have a pressurized chamber in the rock, which is losing gas by seepage. Gas is introduced into the cavity at the same rate at which it is lost through the walls. At time, t = 0, a radioactive tracer gas is introduced into the chamber. We wish to know how far into the rock the radioactivity will have penetrated by a time, $t = \tau$. The average rate of advance of the gas in linear flow in a rock of porosity, f, is

$$\bar{v} = \frac{\bar{Q}}{f}, \text{ hence } D_p = \frac{\bar{Q}\tau}{f}. \tag{6}$$

The pressure gradient must extend at least as far as D_p. The reactor cavity pressure is taken as 6 atm, and that at D_p as 1 atm. Then

$$L = D_p \tag{7}$$

and

$$D_p^{\,2} = \frac{1}{f} \cdot \frac{K\tau\Delta P}{\mu} \quad \text{or} \tag{8}$$

$$D_p = \frac{\sqrt{\dfrac{10^{-2}\,(1200)\,5}{2 \times 10^{-2}}}}{\sqrt{f}} = \frac{55}{\sqrt{f}} \text{ cm}$$

$$= 120 \text{ cm} \qquad f = .20$$

$$= 550 \text{ cm} \qquad f = .01.$$

APPENDIX V

NUCLEAR TEST EXPERIENCE AT NEVADA TEST SITE

There are a number of fundamental differences between an underground nuclear explosion and the (postulated) release of radioactivity from an underground nuclear reactor. Nevertheless, some of the experience gained from the underground test program is applicable also to the problem of safety assurance for underground nuclear power plants. In the following discussion we begin with a review of the physical phenomena involved in the nuclear explosion, with particular attention to those shots which were not fully contained. Conclusions drawn from these data are then applied to the problem of containment of the "maximum credible accident" (MCA), which is conventionally used to determine population exclusion areas for use by the AEC in reactor site evaluation. The MCA considered here is specific to water-cooled reactors. Briefly, such an accident would occur after a rupture somewhere in the coolant system with loss of coolant and subsequent radioactive material release. Radioactive material is then dispersed in the coolant after melting or rupture of fuel elements, the ultimate result being the filling of the vapor containment vessel with superheated steam, gaseous fission products, and liquid and solid aerosols.

Experience at the Nevada Test Site (NTS) shows that the depth of burial (DOB) required for containment of a nuclear explosion is given roughly by $D \sim 400\ W^{1/3}$ where W is yield in kilotons and D is the depth of burial (DOB) in feet. It will be seen later that more is involved than simple cube-root scaling, but this rule gives an idea of the required depths. Hence a 20 kiloton bomb, slightly larger than those used at Hiroshima and Nagasaki in World War II, would be emplaced at about 1000 feet or more to insure containment.

Usually the bomb is placed in a small chamber at the bottom of a shaft. The shaft is then stemmed with alternating layers of sand and gravel. At detonation, a strong shock is driven into the surrounding rock, vaporizing the rock out to a radius of about 20 feet in this example, and melting the rock for some distance beyond. This bubble of rock vapor and steam continues to expand for about 100 msec, when the pressure falls to approximately the overburden pressure (1 psi for every foot of depth-of-burial). The cavity radius at this point is about 100 feet. At about this time the initial shock reaches the ground surface, and reflects as a rarefaction wave. Much of the rock between the shot point and the surface is fractured under this initial shock. If the DOB were not sufficient for containment, the returning rarefaction would allow further upward growth of the cavity, until it reached the surface. Under these circumstances the cavity will break the surface to form a crater, releasing some radioactivity to the atmosphere.

At a depth-of-burial adequate for containment, the cavity stabilizes at its final radius, and the returning rarefaction wave is too weak, and cavity pressure too low, to cause further growth. At this time, the rock vapor has condensed and the cavity is filled primarily with superheated steam, and various non-condensable gases including fission products. The cavity may stand for a few seconds, minutes or even hours, but eventually it will collapse (there are two known exceptions, both in salt). Collapse generally occurs only when the cavity pressure has fallen below ambient overburden pressures. Upon collapse, the entire ceiling falls into the cavity, followed by the progressive collapse of overlying material. If the shot medium is rock, the collapse proceeds upward until the cavity volume is redistributed throughout a column of rubble with decreased bulk density, or until rock is encountered which is sufficiently strong to support itself over a span roughly equal to the final cavity diameter. In this case a small open cavity may be left at the top of the "chimney," or rubble column.

The Hard Hat event left a typical cylindrical chimney of height 350 feet, diameter 140 feet, with a 34-foot void at the top. This event was a 5.9 KT yield in granite at a depth of 940 feet. The permeability of the rubble was estimated as $\sim 10^6$ darcies.

A somewhat different type of collapse occurs in a granular medium such as alluvium, in which the bulk density does not change appreciably upon collapse. In such media, the entire cavity volume is displaced upward upon collapse, to form a subsidence crater at the surface.

There are two ways in which radioactivity has been released inadvertently from underground tests: prompt venting and seepage. Venting is the prompt release (at high pressure) of cavity gases and particulate matter through a more or less direct path to the surface. The few ventings that have been experienced have usually resulted from mechanical failures in stemming or closure devices, especially in tunnels, or in line-of-sight pipes leading into the emplacement chamber.

Seepage is the slow diffusion of the noncondensable cavity gases through the chimney rubble. It may begin from within a few minutes to a few hours after the explosion, and generally continues for a few hours.

From August 1963 (when the limited test-ban treaty was signed) through the end of 1970, the U.S. conducted 225 underground nuclear tests. In 17 events, all at NTS, measurable radioactivity was observed off-site. It should be noted that off-site detection generally implies release of radioactivity of more than 100 curies.

Of these 17 tests, 13 fall into the class discussed above in connection with venting, and are attributed essentially to mechanical failures and man-made discontinuities in the rock. In at least 3 of these events, release occurred through surface ground fissures which were opened by the ground shock and apparently intersected either the cavity or the stemming. The remaining 4 events involved seepage rather than venting, and are not attributable to mechanical failure of closure devices. Seepage in these 4 events was not detected until surface-subsidence had occurred. This delayed seepage was apparently just the upward diffusion of non-condensable radioactive gases through the highly permeable chimney rubble column, which intersected the surface.

The seepage of radioactive gases through the chimney rubble column has been studied rather thoroughly. F. W. Aron (Reference V-1) has reviewed data from 28 contained events (of yields ranging from 1 MT to a fraction of a kiloton). These data pertain to the height to which cavity gases rise in the chimney rubble, and are obtained by post-shot drilling. These test holes are drilled vertically or near-vertically downward into the chimney, one object being to determine the amount of rubble required to quench the contaminated vapors and to stop the displacement and diffusion of the noncondensable gases. The data give either the position of the drill bit when radioactivity is first detected, or the position of the bit when loss of circulation (LOC) of drill fluid occurs (indicating penetration of a highly permeable zone, i.e., the chimney rubble). It was found the LOC either occurs above, or at, the level at which radioactivity is detected, suggesting that the gases do not always penetrate to the top of the rubble chimney. Several factors act to limit seepage, even though the rubble column is usually highly permeable. As already noted, the final cavity pressure is primarily due to superheated steam, and is somewhat greater than ambient (lithostatic) pressure at the time at which the cavity stabilizes. Before collapse occurs, the pressure has presumably fallen below ambient, and the filling of the cavity with rubble removes more steam by condensation. The gases remaining in the rubble can only reach the top of the chimney by a long and tortuous path over cool rock surfaces. Iodines are lost by adsorption and settling out; the remaining gas is primarily xenon and krypton. The pressure (hence seepage rate) of the gas is reduced by heat transfer to the cold rubble, and the shorter-lived isotopes decay to solids and settle out. (Analysis of these data, Reference V-1) and a review of those 17 events which were not contained, show that burial at a depth given by the rule, $D = 400\ W^{1/3}$ is adequate up to about 10 KT and increasingly conservative above 10 KT. That is, containment of seepage is not governed by cube-root scaling. The reason for this is as follows: both the amount of vapor in the cavity, and the volume of the stabilized cavity are proportional to yield, hence the final cavity pressure is nearly independent of yield (aside from effects of overburden pressure). However, test devices are buried roughly according to cube-root scaling; hence the gases from a 1 MT bomb, at perhaps twice the initial pressure, must seep 10 times ($1000^{1/3}$) as far as those resulting from a kiloton bomb, to escape at the surface. Hence the $W^{1/3}$ rule, while appropriate for avoidance of blast cratering, is conservative from the standpoint of containment of seepage from the larger bombs.

For an underground reactor the maximum credible accident (MCA) would result in the filling of the reactor chamber (lined with steel or concrete) with steam and fission products, at initial steam pressures of the order of a hundred psi. Data obtained in the AJAX Nuclear Test event seems pertinent. AJAX (11/11/66) was a 6.5 KT shot at a depth of 240 meters in alluvium (12% water by weight) at Yucca Flat of the Nevada Test Site (NTS). Pressure transducers were placed in a pipe extending upward from the working point, yielding cavity pressure measurements from about 15 seconds after zero time until collapse at 10 minutes. It was estimated that the stabilized cavity held 600 tons of steam, at an initial measured pressure of 44 bars. Analysis of the cavity pressure history (Reference V-2) indicated that the cavity cooled rapidly by heat-transfer to the cavity walls, according to what one would expect for the cooling of a body whose thermal conductivity is large compared to that of the surroundings. In this case, the temperature history is given by:

$$T - T_{\infty} = (T_o - T_{\infty}) \exp(-hA/nC_v)t$$

where:

T	=	temperature at time t
h	=	surface heat transfer coefficient
A	=	surface area
nC_v	=	heat capacity of cooling mass
	=	total moles steam x molar heat capacity
T_{∞}	=	ambient temperature

Figure V-1 shows the comparison between measured and calculated pressures. The pressures are calculated (using the gas law to relate P and T) by use of the foregoing equation with the estimated values of A and h. The best fit corresponds to a cavity thermal time constant:

$$\tau = \frac{nC_v}{hA} = 2.8 \text{ minutes}$$

The pressure drop beginning at about 7 minutes corresponds to condensation of the steam. Figure V-2 (Reference V-2) shows the calculated temperature and phase concentrations in the cavity as functions of time. The rate of condensation after 7 minutes implies a very high temperature gradient in the cavity walls, which must have spalled and flaked into the cavity, increasing the cavity surface area and accelerating the rate of cooling.

The mass of steam in the AJAX cavity (600 tons) is of the order of that which could be produced in the MCA for a 1000 MW_e reactor, and the surface area of the cavity ($10^5 ft^2$) is also comparable to that of the corresponding underground reactor chamber. Hence one would expect a comparable time constant for the reactor chamber (ignoring any provision for steam suppression).

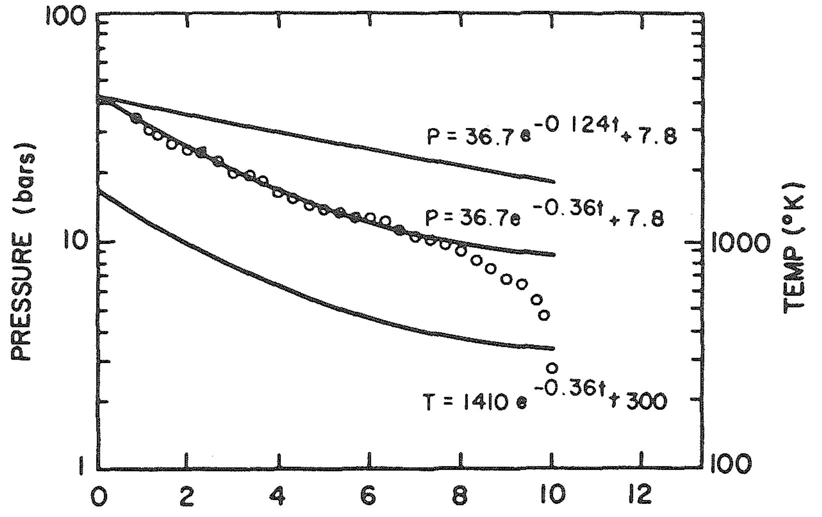

Figure V-1. Cavity Pressure-Temperature History, AJAX Event

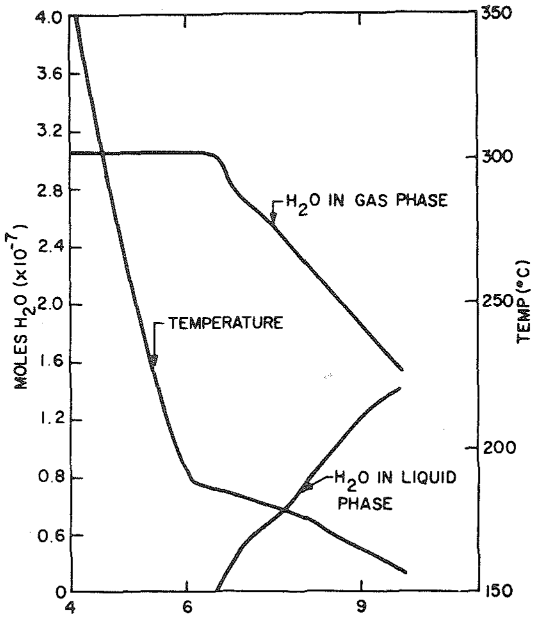

Figure V-2. Calculated Temperature and Phase Distribution of Water in AJAX Cavity

REFERENCES

V-1 "Phenomenology and Containment of Underground Nuclear Explosions," L. S. Germain and J. S. Kahn, UCRL 50482, November 1968. (Arons work is discussed, original reference not available.)

V-2 "Time History of Cavity Pressure and Temperature Following a Nuclear Detonation in Alluvium," C. W. Olsen, Journal of Geophysical Research 72, October 15, 1967.

APPENDIX VI

COST DETAIL

In preparing this report several different plant configurations were costed based on the type of reactor, quality of rock, and seismic loading. Twenty-two separate elements of each plant configuration were costed for excavation, concrete lining, and steel liner. The following summary sheets show the costs for each of these items and the excavation-construction subtotals for each configuration and site condition.

PWR TYPE RQD= 85

	Str. Class	Net Dimensions L_{ft}	W_{ft}	H_{ft}	Wall - T Ft D	D1	Excavation L_{ft}	W_{ft}	H_{ft}	Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost \$M	Conc. & Surf. Prep. Cost \$M	Steel Liner Cost \$M	Total Cost \$M
Reactor	I	120	60	135	1 1/2	4	128	102	146	59943	53104	6839	1.199	.684	1.406	3.289
- Instrumentation	I	30	10	10	*	*	31	13	12	161	143	18	.003	.002	-	.005
Turbine Generator	II	355	90	100	3	1	357	117	117	164600	152333	12267	3.292	1.227	-	4.519
Nuclear Auxiliary	I	260	50	90	1	1 3/4	264	76	98	64005	59486	4519	1.280	.452	-	1.732
Control and Miscellaneous	I	230	60	30	1 1/2	*	231	68	41	21640	19671	1968	.433	.197	-	.630
Feed Water Heaters	II	100	40	45	1/2	*	101	52	51	9165	8823	342	.183	.034	-	.217
Transformer Room	II	50	30	50	*	*	51	43	54	4095	3952	143	.082	.014	-	.096
Cooling Water Tunnels (2)	II	2000	20	Dia	*	*				46500	43360	3140	.930	.314	-	1.244
Access Tunnel or Shaft	I	250	40 x	40	1	*				14817	14067	750	.889	.075	-	.964
Control Room Access Tunnel	I	200	20	20	*	*	200	26	23	3959	3747	213	.079	.021	-	.100
Nuclear Auxiliary Access Tunnel	I	220	20	20	*	*	200	26	23	4350	4117	233	.087	.023	-	.110
Reactor Access & Water Tunnel	I	160	30	36	1/2	*	160	40	41	8672	8299	374	.173	.037	-	.211
Transformer Access Tunnel	II	80	25	25	*	*	80	32	29	2522	2403	118	.050	.012	-	.062
Reactor Steam Tunnel	II	160	10	10	*	*	160	13	12	828	745	83	.017	.008	-	.025
Feed Water Heater Tunnels	II	160	14	12	*	*	160	18	14	2710	2492	218	.054	.022	-	.076
	II	80	6	6	*	*	80	8	7	318	268	50	.006	.006	-	.012
Reactor Access & Water Tunnel	I	160	7	7	*	*	160	9	9	422	364	58	.008	.006	-	.014
Fuel Transfer Tunnel	I	250	4	Dia	*	*				100	80	20	.002	.002	-	.004
Air Shaft	I	250	10 x	10	*	*				917	727	190	.055	.019	-	.074
Elevator Shafts (3)	II	250	10 x	10	*	*				2750	2180	570	.165	.057	-	.222
Power In Shaft	II	250	10	Dia	*	*				733	633	100	.044	.010	-	.054
Power Out Shaft	II	250	10	Dia	*	*				733	633	100	.044	.010	-	.054
Fuel Transfer Shaft	I	250	10 x	10	*	*				917	727	190	.055	.019	-	.074
							TOTALS			414857	382354	32503	9.130	3.251	1.406	13.748

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

BWR TYPE - RECONFIGURED RQD = 85

	Str. Class	Net Dimensions L ft	W ft	H ft	Wall - T Ft D	D1	Excavation L ft	W ft	H ft	Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
Reactor	I	155	65	65	2	1	157	83	77	34482	31467	3015	.690	.302	1.062	2.053
- Control Drive (2)	I	50	30	40	1/2	*	51	41	45	6414	6098	316	.128	.032	-	.160
Turbine Generator	II	415	100	100	3 3/4	1	417	127	120	213877	195741	18136	4.278	1.814	-	6.091
Nuclear Auxiliary	I	255	50	90	1	1 3/4	259	76	98	62803	58364	4440	1.256	.444	-	1.700
Control and Miscellaneous	I	320	60	30	1 1/2	*	321	68	41	29820	27171	2649	.596	.265	-	.861
Feed Water Heaters	II	100	40	45	1/2	*	101	52	51	9165	8823	342	.183	.034	-	.217
Transformer Room	II	50	30	50	*	*	51	43	54	4095	3952	143	.082	.014	-	.096
Cooling Water Tunnels (2)	II	2000	20	Dia	*	*				46500	43360	3140	.930	.314	-	1.244
Access Tunnel or Shaft	I	250	40	40	1	*				14817	14067	750	.889	.075	-	.964
Control Room Access Tunnel	I	200	20	20	*	*	200	26	23	3959	3747	213	.079	.021	-	.100
Nuclear Auxiliary Access Tunnel	I	220	20	20	*	*	220	26	23	4350	4117	233	.087	.023	-	.110
Reactor Access Tunnel	I	160	30	36	1/2	*	160	40	41	8672	8299	374	.173	.037	-	.211
Transformer Access Tunnel	II	80	25	25	*	*	80	32	29	2522	2403	118	.050	.012	-	.062
Reactor Steam Tunnel	II	160	10	10	*	*	160	13	12	828	745	83	.017	.008	-	.025
Feed Water Heater Tunnels	II	160	14	12	*	*	160	18	14	2710	2496	218	.054	.022	-	.076
	II	80	6	6	*	*	80	8	7	318	268	50	.006	.006	-	.012
Reactor Water Tunnel	I	160	7	7	*	*	16	9	9	422	364	58	.008	.006	-	.014
Fuel Transfer Tunnel	I	250	4	Dia	*	*				100	80	20	.002	.002	-	.004
Air Shaft	I	250	10 x	10	*	*				917	727	190	.055	.019	-	.074
Elevator Shafts (3)	II	250	10 x	10	*	*				2750	2180	570	.165	.057	-	.222
Power In Shaft	II	250	10 D	ia	*	*				733	633	100	.044	.010	-	.054
Power Out Shaft	II	250	10 D	ia	*	*				733	633	100	.044	.010	-	.054
Fuel Transfer Shaft	I	250	10 x	10	*	*				917	727	190	.055	.019	-	.074
Reactor Well	I	80	65 D	ia	1	1	81	67	67	10446	9701	745	.627	.074	.326	1.027
							TOTALS			462350	426163	36193	10.498	3.620	1.388	15.504

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

BWR TYPE - MINIMUM MODIFICATION RQD = 85

	Str. Class	Net Dimensions L_{ft}	Net Dimensions W_{ft}	Net Dimensions H_{ft}	Wall - T Ft D	Wall - T Ft D1	Excavation L_{ft}	Excavation W_{ft}	Excavation H_{ft}	Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
Reactor	I	215	75	180	2	7	229	134	193	184418	158819	25599	3.688	2.560	-	6.248
Turbine Generator	II	415	100	100	3 3/4	1	417	127	120	213877	195741	18136	4.278	1.814	-	6.091
Nuclear Auxiliary	I	155	50	90	1	1 3/4	159	76	98	38774	35926	2848	.775	.285	-	1.060
Control and Miscellaneous	I	245	60	30	1 1/2	*	246	68	41	23003	20921	2082	.460	.208	-	.668
Feed Water Heaters	II	100	40	45	1/2	*	101	52	51	9165	8823	342	.183	.034	-	.217
Transformer Room	II	50	30	50	*	*	51	43	54	4095	3952	143	.082	.014	-	.096
Cooling Water Tunnels (2)	II	2000	20	Dia	*	*				46500	43360	3140	.930	.314	-	1.244
Access Tunnel or Shaft	I	250	40	40	1	*				14817	14067	750	.889	.075	-	.964
Control Room Access Tunnel	I	220	20	20	*	*	220	26	23	4350	4117	233	.087	.023	-	.110
Nuclear Auxiliary Access Tunnel	I	220	20	20	*	*	220	26	23	4350	4117	233	.087	.023	-	.110
Reactor Access Tunnel	I	160	30	36	1/2	*	160	40	41	8672	8299	374	.173	.037	-	.211
Transformer Access Tunnel	II	80	25	25	*	*	80	32	29	2522	2403	118	.050	.012	-	.062
Reactor Steam Tunnel	II	160	10	10	*	*	160	13	12	828	745	83	.017	.008	-	.025
Feed Water Heater Tunnels	II	160	14	12	*	*	160	18	14	2710	2492	218	.054	.022	-	.076
	II	80	6	6	*	*	80	8	7	318	268	50	.006	.006	-	.012
Reactor Access & Water Tunnel	I	160	7	7	*	*	160	9	9	422	364	58	.008	.006	-	.014
Fuel Transfer Tunnel																
Air Shaft	I	250	10	x 10	*	*				917	727	190	.055	.019	-	.074
Elevator Shafts (3)	II	250	10 x	10	*	*				2750	2180	570	.165	.057	-	.222
Power In Shaft	II	250	10	Dia	*	*				733	633	100	.044	.010	-	.054
Power Out Shaft	II	250	10	Dia	*	*				733	633	100	.044	.010	-	.054
Fuel Transfer Shaft	I	250	10	x 10	*	*				917	727	190	.055	.019	-	.074
							TOTALS			564871	509314	55557	12.130	5.556	-	17.686

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

PWR TYPE RQD = 60

	Str. Class	Net Dimensions L_{ft}	Net Dimensions W_{ft}	Net Dimensions H_{ft}	Wall - T Ft D	Wall - T Ft D1	Excavation L_{ft}	Excavation W_{ft}	Excavation H_{ft}	Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
Reactor	I	120	60	135	9 1/4	7 3/4	136	109	161	68840	46073	22767	1.377	2.277	1.406	5.060
- Instrumentation	I	30	10	10	*	*	31	13	12	161	118	43	.003	.004	-	.007
Turbine Generator	II	355	90	100	12	2 3/4	321	121	135	191115	142222	48893	3.822	4.889	-	8.711
Nuclear Auxiliary	I	260	50	90	5 1/2	3 2/3	267	80	107	71887	56050	15837	1.438	1.584	-	3.022
Control and Miscellaneous	I	230	60	30	4 1/3	3/4	232	69	46	24814	18498	6316	.496	.632	-	1.128
Feed Water Heaters	II	100	40	45	2 1/3	1/2	101	52	55	9777	8164	1613	.196	.161	-	.357
Transformer Room	II	50	30	50	1 2/3	1/2	51	44	57	4279	3562	717	.086	.072	-	.158
Cooling Water Tunnels (2)	II	2000	20 Dia		1/2	*				51312	46658	4654	1.026	.968	-	1.994
Access Tunnel or Shaft	I	250	40 x	40	2 1/2	1	252	52	50	21613	17852	3761	1.296	.376	-	1.672
Control Room Access Tunnel	I	200	20	20	3/4	*	201	26	24	4086	3451	635	.082	.064	-	.146
Nuclear Auxiliary Access Tunnel	I	220	20	20	3/4	*	221	26	24	4490	3795	694	.090	.069	-	.159
Reactor Access Tunnel	I	160	30	36	1 1/2	3/4	162	41	43	9221	7791	1430	.184	.143	-	.327
Transformer Access Tunnel	II	80	25	25	3/4	*	81	32	30	2585	2221	364	.052	.036	-	.088
Reactor Steam Tunnel	II	160	10	10	*	*	161	13	12	828	633	195	.017	.02	-	.037
	II	160	14	12	*	*	161	18	14	2710	2210	502	.054	.050	-	.104
Feed Water Heater Tunnels	II	80	6	6	*	*	81	8	7	318	196	122	.006	.012	-	.018
Reactor Access & Water Tunnel	I	160	7	7	*	*	161	9	9	422	284	138	.008	.014	-	.022
Fuel Transfer Tunnel	I	250	4 Dia		*	*				100	80	20	.002	.008	-	.010
Air Shaft	I	250	10 x	10	*	*				917	727	190	.055	.039	-	.094
Elevator Shafts (3)	II	250	10 x	10	*	*				2750	2180	570	.165	.117	-	.282
Power In Shaft	II	250	10 Dia		*	*				733	633	100	.044	.018	-	.062
Power Out Shaft	II	250	10 Dia		*	*				733	633	100	.044	.018	-	.062
Fuel Transfer Shaft	I	250	10 x	10	*	*				917	727	190	.055	.039	-	.094
							TOTALS			474608	364758	109851	10.598	11.610	1.406	23.614

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

BWR TYPE- RECONFIGURED RQD= 60

	Str. Class	Net Dimensions L_{ft}	Net Dimensions W_{ft}	Net Dimensions H_{ft}	Wall - T Ft D	Wall - T Ft D1	Excavation L_{ft}	Excavation W_{ft}	Excavation H_{ft}	Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
Reactor	I	155	65	65	6 3/4	2 1/2	160	86	87	39263	28770	10492	0.785	1.049	1.062	2.896
- Control Drive (2)	I	50	30	40	1 2/3	3/4	52	42	47	6808	5492	1318	0.136	.132	-	.268
Turbine Generator	II	415	100	100	14 1/4	2 3/4	421	131	141	252400	182824	69577	5.048	6.958	-	12.006
Nuclear Auxiliary	I	255	50	90	5 1/2	3 2/3	262	80	107	70534	54956	15578	1.411	1.558	-	2.969
Control and Miscellaneous	I	320	60	30	4 1/3	3/4	322	69	46	34237	25746	8491	.685	.849	-	1.534
Feed Water Heaters	II	100	40	45	2 1/3	1/2	101	52	55	9777	8164	1613	.196	.161	-	.357
Transformer Room	II	50	30	50	1 2/3	1/2	51	44	57	4279	3562	717	.086	.072	-	.158
Cooling Water Tunnels (2)	II	2000	20	Dia	1/2	*				51312	46658	4654	1.026	.968	-	1.994
Access Tunnel or Shaft	I	250	40	40	2 1/2	1	252	52	50	21613	17852	3761	1.296	.376	-	1.672
Control Room Access Tunnel	I	200	20	20	2/3	*	201	26	24	4062	3455	607	.081	.061	-	.142
Nuclear Auxiliary Access Tunnel	I	220	20	20	2/3	*	221	26	24	4463	3799	664	.089	.066	-	.155
Reactor Access Tunnel	I	160	30	36	1 1/2	3/4	162	41	43	9221	7791	1430	.184	.143	-	.327
Transformer Access Tunnel	II	80	25	25	3/4	*	81	32	30	2585	2221	364	.052	.036	-	.088
Reactor Steam Tunnel	II	160	10	10	*	*	161	13	12	828	633	195	.017	.020	-	.037
	II	80	6	6	*	*	81	8	7	318	196	122	.006	.012	-	.018
Feed Water Heater Tunnels	II	160	14	12	*	*	161	18	14	2710	2210	502	.054	.050	-	.104
Reactor Water Tunnel	I	160	7	7	*	*	161	9	9	422	284	138	.008	.014	-	.022
Fuel Transfer Tunnel	I	250	4	Dia	*	*				100	80	20	.002	.008	-	.010
Air Shaft	I	250	10 x	10	*	*				917	727	190	.055	.039	-	.094
Elevator Shafts (3)	II	250	10 x	10	*	*				2750	2180	570	.165	.117	-	.282
Power In Shaft	II	250	10	Dia	*	*				733	633	100	.044	.018	-	.062
Power Out Shaft	II	250	10	Dia	*	*				733	633	100	.044	.018	-	.062
Fuel Transfer Shaft	I	250	10 x	10	*	*				917	727	190	.055	.039	-	.094
Reactor Well	I	80	65	Dia	1	1				10446	9701	745	.627	.107	.326	1.060
							TOTALS			531428	409294	122138	12.152	12.871	1.388	26.411

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

BWR TYPE - MINIMUM MODIFICATION RQD = 60

	Str. Class	Net Dimensions			Wall - T Ft		Excavation			Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
		L_{ft}	W_{ft}	H_{ft}	D	D1	L_{ft}	W_{ft}	H_{ft}							
Reactor	I	215	75	180	15 1/3	13 2/3	242	147	220	220550	138436	82114	4.411	8.211	-	12.622
Turbine Generator	II	415	100	100	14 1/4	2 3/4	421	131	141	252400	182824	69577	5.048	6.958	-	12.006
Nuclear Auxiliary	I	155	50	90	5 1/2	3 2/3	162	80	107	43473	33069	10403	0.869	1.040	-	1.909
Control and Miscellaneous	I	245	60	30	4 1/3	3/4	247	69	46	26384	19706	6679	0.528	0.668	-	1.196
Feed Water Heaters	II	100	40	45	2 1/3	1/2	101	52	55	9777	8164	1613	0.196	0.161	-	0.357
Transformer Room	II	50	30	50	1 2/3	1/2	51	44	57	4279	3562	717	0.086	0.072	-	0.158
Cooling Water Tunnels (2)	II	2000	20	Dia	1/2	*				51312	46658	4654	1.026	0.968	-	1.994
Access Tunnel or Shaft	I	250	40	40	2 1/2	1	252	52	50	21613	17852	3761	1.296	0.376	-	1.672
Control Room Access Tunnel	I	220	20	20	2/3	*	221	26	24	4463	3799	664	0.089	0.066	-	0.155
Nuclear Auxiliary Access Tunnel	I	220	20	20	2/3	*	221	26	24	4463	3799	664	0.089	0.066	-	0.155
Reactor Access Tunnel	I	160	30	36	1 1/2	3/4	162	41	43	9221	7791	1430	0.184	0.143	-	0.327
Transformer Access Tunnel	II	80	25	25	3/4	*	81	32	30	2585	2221	364	0.052	0.036	-	0.088
Reactor Steam Tunnel	II	160	10	10	*	*	161	13	12	828	633	195	0.017	0.020	-	0.037
Feed Water Heater Tunnels	II	160	14	12	*	*	161	18	14	2710	2210	502	0.054	0.050	-	0.104
	II	80	6	6	*	*	81	8	7	318	196	122	0.006	0.012	-	0.018
Reactor Access & Water Tunnel	I	160	7	7	*	*	161	9	9	422	284	138	0.008	0.014	-	0.022
Fuel Transfer Tunnel																
Air Shaft	I	250	10	x 10	*	*				917	727	190	.055	.039	-	0.094
Elevator Shafts (3)	II	250	10	x 10	*	*				2750	2180	570	.165	.117	-	0.282
Power In Shaft	II	250	10	Dia	*	*				733	633	100	.044	.018	-	0.062
Power Out Shaft	II	250	10	Dia	*	*				733	633	100	.044	.018	-	0.062
Fuel Transfer Shaft	I	250	10	x 10	*	*				917	727	190	.055	.039	-	0.094
							TOTALS			660848	476104	184747	14.322	19.092	-	33.414

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

PWR TYPE RQD = 40

	Str. Class	Net Dimensions L_{ft}	Net Dimensions W_{ft}	Net Dimensions H_{ft}	Wall - T Ft D	Wall - T Ft D1	Excavation L_{ft}	Excavation W_{ft}	Excavation H_{ft}	Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
Reactor	I	120	60	135	15	12 2/3	145	119	173	78110	34888	43222	1.562	4.322	1.406	7.290
- Instrumentation	I	30	10	10	*	*	31	13	12	161	68	93	.003	.009	-	.012
Turbine Generator	II	355	90	100	19 3/4	4 1/2	364	124	151	214613	119182	95431	4.292	9.543	-	13.835
Nuclear Auxiliary	I	260	50	90	9	6	272	85	114	79527	49699	29827	1.591	2.983	-	4.574
Control and Miscellaneous	I	230	60	30	7	1 1/4	233	70	52	27864	16309	11555	.557	1.155	-	1.712
Feed Water Heaters	II	100	40	45	3 3/4	1	102	53	58	10380	7101	3279	.208	.328	-	.536
Transformer Room	II	50	30	50	2 2/3	1	52	45	59	4489	2951	1538	.090	.154	-	.244
Cooling Water Tunnels (2)	II	2000	20 Dia		1	*				56316	46542	9773	1.126	.977	-	2.103
Access Tunnel or Shaft	I	250	40 x	40	4	1 1/2	253	53	53	23131	15946	7185	1.387	.719	-	2.106
Control Room Access Tunnel	I	200	20	20	1	1/2	201	26	25	4214	2860	1355	.084	.135	-	.219
Nuclear Auxiliary Access Tunnel	I	220	20	20	1	1/2	221	26	25	4630	3150	1480	.093	.148	-	.241
Reactor Access Tunnel	I	160	30	36	2 1/2	1	162	41	45	9695	6801	2894	.194	.289	-	.483
Transformer Access Tunnel	II	80	25	25	1 1/2	*	81	32	31	2698	1844	855	.054	.085	-	.139
Reactor Steam Tunnel	II	160	10	10	*	*	161	13	12	828	410	418	.017	.042	-	.059
Feed Water Heater Tunnels	II	160	14	12	1/2	*	161	18	15	2768	1630	1138	.056	.104	-	.160
	II	80	6	6	*	*	81	8	7	318	54	264	.006	.026	-	.032
Reactor Access & Water Tunnel	I	160	7	7	*	*	161	9	9	422	124	297	.008	.030	-	.038
Fuel Transfer Tunnel	I	250	4	Dia	*	*				100	80	20	.002	.020	-	.022
Air Shaft	I	250	10 x	10	*	*				917	727	190	.055	.079	-	.134
Elevator Shafts (3)	II	250	10 x	10	*	*				2750	2180	570	.165	.237	-	.402
Power In Shaft	II	250	10	Dia	*	*				733	633	100	.044	.057	-	.101
Power Out Shaft	II	250	10	Dia	*	*				7733	633	100	.044	.057	-	.101
Fuel Transfer Shaft	I	250	10 x	10	*	*				917	727	190	.055	.079	-	.134
							TOTALS			526314	314539	211774	11.693	21.578	1.406	34.677

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

BWR TYPE-RECONFIGURED RQD= 40

	Str. Class	Net Dimensions			Wall - T Ft		Excavation			Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
		L_{ft}	W_{ft}	H_{ft}	D	D1	L_{ft}	W_{ft}	H_{ft}							
Reactor	I	155	65	145	11	4	163	89	95	43663	24296	19367	.873	1.937	1.062	3.872
- Control Drive (2)	I	50	30	40	2 2/3	1 1/3	53	43	49	7214	4468	2746	.144	.274	-	.418
Turbine Generator	II	415	100	100	23 1/3	4 2/3	424	134	159	287014	139878	147136	5.740	14.714	-	20.454
Nuclear Auxiliary	I	255	50	90	9	6	267	85	114	78027	48680	29347	1.561	2.935	-	4.496
Control and Miscellaneous	I	320	60	30	7	1 1/4	323	70	52	38480	22960	15520	.770	1.552	-	2.322
Feed Water Heaters	II	100	40	45	3 3/4	1	102	53	58	10380	7101	3279	.208	.328	-	.536
Transformer Room	II	50	30	50	2 2/3	1	52	45	59	4489	2951	1538	.090	.154	-	.244
Cooling Water Tunnels	II	2000	20	Dia	1	*				56316	46542	9773	1.126	.977	-	2.103
Access Tunnels or Shaft	I	250	40	40	4	1 1/2	253	53	53	23131	15946	7185	1.387	.719	-	2.106
Control Room Access Tunnel	I	200	20	20	1	1/2	201	26	25	4214	2860	1355	.084	.135	-	.219
Nuclear Auxiliary Access Tunnel	I	220	20	20	1	1/2	221	26	25	4630	3150	1480	.093	.148	-	.241
Reactor Access Tunnel	I	160	30	36	2 1/2	1	162	41	45	9695	6801	2894	.194	.289	-	.483
Transformer Access Tunnel	II	80	25	25	1 1/2	*	81	32	31	2698	1844	855	.054	.085	-	.139
Reactor Steam Tunnel	II	160	10	10	*	*	161	13	12	828	410	418	.017	.042	-	.059
	II	160	14	12	1/2	*	161	18	15	2768	1630	1138	.056	.104	-	.160
Feed Water Heater Tunnels	II	80	6	6	*	*	81	8	7	318	54	264	.006	.026	-	.032
Reactor Water Tunnel	I	160	7	7	*	*	161	9	9	422	124	297	.008	.030	-	.038
Fuel Transfer Tunnel	I	250	4	Dia	*	*				100	80	20	.002	.020	-	.022
Air Shaft	I	250	10 x	10	*	*				917	727	190	.055	.079	-	.134
Elevator Shafts (3)	II	250	10 x	10	*	*				2750	2180	570	.165	.237	-	.402
Power In Shaft	II	250	10	Dia	*	*				733	633	100	.044	.057	-	.101
Power Out Shaft	II	250	10	Dia	*	*				733	633	100	.044	.057	-	.101
Fuel Transfer Shaft	I	250	10 x	10	*	*				917	727	190	.055	.079	-	.134
Reactor Well	I	80	65	Dia	1 1/3	1 1/3				10656	9835	821	.639	.180	.326	1.145
							TOTALS			591093	344510	246583	13.415	25.158	1.388	39.961

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

BWR TYPE - MINIMUM MODIFICATION RQD = 40

	Str. Class	Net Dimensions			Wall - T Ft		Excavation			Excavation Vol. Yd^3	Net Vol. Yd^3	Conc. Vol. Yd^3	Excavation Cost $M	Conc. & Surf. Prep. Cost $M	Steel Liner Cost $M	Total Cost $M
		L_{ft}	W_{ft}	H_{ft}	D	D1	L_{ft}	W_{ft}	H_{ft}							
Reactor	I	215	75	180	25	22 1/2	260	165	239	258664	113826	144838	5.173	14.484	-	19.657
Turbine Generator	II	415	100	100	23 1/3	4 2/3	424	134	159	287014	139878	147136	5.740	14.714	-	20.454
Nuclear Auxiliary	I	155	50	90	9	6	167	85	114	48027	28280	19747	.961	1.975	-	2.936
Control and Miscellaneous	I	245	60	30	7	1 1/4	248	70	52	29633	17418	12216	.593	1.222	-	1.815
Feed Water Heaters	II	100	40	45	3 3/4	1	102	53	58	10380	7101	3279	.208	.328	-	.536
Transformer Room	II	50	30	50	2 2/3	1	52	45	59	4489	2951	1538	.090	.154	-	.244
Cooling Water Tunnels	II	2000	20	Dia	1	*				56316	46542	9773	1.126	.977	-	2.103
Access Tunnel or Shaft	I	250	40	40	4	1 1/2	253	53	53	23131	15946	7185	1.387	.719	-	2.106
Control Room Access Tunnel	I	220	20	20	1	1/2	221	26	25	4630	3150	1480	.093	.148	-	.241
Nuclear Auxiliary Access Tunnel	I	220	20	20	1	1/2	221	26	25	4630	3150	1480	.093	.148	-	.241
Reactor Access Tunnel	I	160	30	36	2 1/2	1	162	41	45	9695	6801	2894	.194	.289	-	.483
Transformer Access Tunnel	II	80	25	25	1 1/2	*	81	32	31	2698	1844	855	.054	.085	-	.139
Reactor Steam Tunnel	II	160	10	10	*	*	161	13	12	828	410	418	.017	.042	-	.059
Feed Water Heater Tunnels	II II	160 80	14 6	12 6	1/2 *	 *	161 81	18 8	15 7	2768 318	1630 54	1138 264	.056 .006	.104 .026	- -	.160 .032
Reactor Access & Water Tunnel	I	160	7	7	*	*	161	9	9	422	124	297	.008	.030	-	.038
Fuel Transfer Tunnel																
Air Shaft	I	250	10	x 10	*	*				917	727	190	.055	.079	-	.134
Elevator Shafts (3)	II	250	10	x 10	*	*				2750	2180	570	.165	.237	-	.402
Power In Shaft	II	250	10	Dia	*	*				733	633	100	.044	.057	-	.101
Power Out Shaft	II	250	10	Dia	*	*				733	633	100	.044	.057	-	.101
Fuel Transfer Shaft	I	250	10	x 10	*	*				917	727	190	.055	.079	-	.134
							TOTALS			749693	394005	355688	16.162	35.954	-	52.116

* minimum liner (4" thick or equiv.)

D, roof arch thickness
D1, wall arch thickness

APPENDIX VII

CALIFORNIA COAST SITING

The location of potential underground power plant sites was not included in the scope of the present study and was not undertaken. The number and location of suitable sites is, of course, a very important question. The following discussion is based upon a cursory review of readily available reports and is not presented as a complete review of this issue.

Several areas along the California Coast have been selected as potential underground power plant siting areas in the map and literature study, "Office Study of Underground Nuclear Power Plant Siting Along the California Coast" (Reference VII-1). It should be noted that no field investigation has been made and, therefore, little is known about the rock material properties in the areas discussed. The following discussion is extracted from this reference. In each case the area is identified by the end points of a segment of the coast and the length of the segment.

One of the better areas appears to be about 10 miles south of Monterey between Notley's Landing and Point Lobos. The criteria evaluated in Reference VII-1 and the corresponding comments include:

Notley's Landing to Point Lobos State Park - Eight Miles

1. Zero to three miles from coast: on shoreline.
2. Located on hard, competent rock: granite block is 30 to 50 square miles in area.
3. Only two faults are in the vicinity and these are spaced seven miles apart. These faults do not appear to be a problem.
4. Access up to one-half to one mile inland is good by way of Highway One.

5. Land here is privately owned and theoretically can be purchased.

6. Moss Landing Power Plant is to the north about 15 miles with major transmission lines through the area by 1990. This area is fairly close to San Francisco, a major load center.

7. The population density of the area is very low, though many people use parts of the region for recreational purposes.

8. Granite faces rise from the beach at a rate of up to 1,500 feet per mile: more than sufficient to get a reasonable ground cover at minimum excavation.

Santa Lucia quartz diorite and Sur Series metamorphic rocks are found in this area. It is an area of homogeneous granite with no known faults over a large area. The extent of weathe ring is not known. However, the depth of burial of the power plant should put the roof of the excavation well into competent rock to provide a strong rock arch. This site is probably a dry site with large joint spacing and no fracturing.

An area which may be suitable but less desirable than the above area is the Point Arguello site described as follows.

<u>Point Arguello and Point Pedernales - One Mile</u>

1. The rock being looked at is from three-quarters to three miles inland.

2. The rock type here is rhyolite. It is very possible that this is suitable rock, but the depth is not known. A closer study should be made of the area.

3. The Honda Fault runs from one to three miles north of the area, but it should not be a problem.

4. One railroad runs from one-half to three miles from the area and an improved light duty road is close by.

5. This land is privately owned.

6. The Mesa Power Plant is close to the area and the North Coast Power Plant is forecast by 1990.

7. The population density of this area is low.

8. The relief of the area is sufficient, but it is situated in fairly rugged land.

Ryolite is an igneous intrusive rock and is similar to granite but tends to have finer grains and is more jointed or fractured. Due to the jointing and fractures, an underground excavation in this area could either be wet or dry depending on the amount of rainfall and severity of jointing. If water is encountered, the entire cavity may have to be sealed with concrete walls, roof and floor.

An area of weak rock that might be considered for underground siting lies in the area from Oceanside to San Clemente. It is described in the siting report as follows.

Oceanside to San Clemente - 22 Miles

1. Geology

   From 8 to 12 miles north of Oceanside there is a small four-mile strip of Miocene marine rock that has a high possibility of being hard. It should be looked into with more detail. Around San Clemente there are more of the Miocene marine rocks. There are few apparent faults in these areas but two head for the possible sites and disappear under recent sediments.

2. Access and Terrain

   The potential siting area of this region (8 to 12 miles north of Oceanside) is along the beach on the San Onofre Bluff with a relief of about 1,000 feet per mile. Access in this area is very good. Only in a few places is the terrain very rough, and many developed roads run close by.

3. Ownership, Population, and Power Distribution

The area between Oceanside and San Clemente is mainly occupied by Camp Pendleton U.S. Marine Corps.

U.S. Government owned land is:

T5W	R11S[1]	SB[2]
T5W	R10S	
T6W	R10S	
T6W	R9S	
T7W	R8S	

Population:

Oceanside	40,686
San Clemente	16,462
Camp Pendleton	?

San Onofre Nuclear Power Plant is located just south of San Clemente.

Main transmission lines from the San Onofre plant run north to Los Angeles and south to San Diego.

This week sandstone may be self supporting for small excavations such as small diameter tunnels. However, it is felt that any large excavations would have to be lined with a heavy liner.

Potential Siting Areas in San Diego County

In addition to the siting areas along the immediate coast identified in Reference VII-1 there are igneous intrusive masses located approximately 3 - 6 miles from the coastline in an area from Cardiff-by-the Sea to Camp Pendleton north of Oceanside which is approximately 20 miles of ocean frontage. The surface geology of some of these areas has been identified by Michael P. Kennedy for his doctoral dissertation at the University of California, Riverside. On the California State Geology Maps scale 1 to 250,000 the rock is identified as Mesozoic granitic rocks such as granite, granodiorite, tonalite and diorite as well as Jura-Trias meta volcanic rocks. Mr. Kennedy has identified those in the Agua Hedionda area south of Carlsbad as quartz diorite, Jurassic metamorphic volcanics and Tertiary andesite. The volcanics are moderately too highly fractured; however, one outcrop of Tertiary andesite which forms the hill Cerro de la Calavera has been identified by Mr. Kennedy as very resistant and unfractured at the surface. The area at the surface is approximately 1500 feet in an east-west direction and 1000 feet in a north-south direction. The elevation at the top of the hill is 513 feet above mean sea level. A location such as this might provide sufficient area and cover for a plant of the size described in this report. This particular location is approximately 4-1/2 miles from the coastline in the north branch of the Agua Hedionda Canyon.

The nearest subdivision is approximately 3/4 of a mile away. A major transmission line crosses the northern edge of the site leading from the Carlsbad Fossil Fuel Power Plant owned by San Diego Gas and Electric. Access to the site would be excellent by developing existing dirt roads in Agua Hedionda Canyon. It would also be feasible to bring a railroad spur from the Santa Fe Los Angeles-San Diego main line. These sites in San Diego county may merit additional study and investigation along with several of the sites mentioned in "Office Study of Underground Nuclear Power Plant Siting Along the California Coast."

REFERENCES

VII-1 Seamount, D., "Office Study of Underground Nuclear Power Plant Siting Along the California Coast," The Aerospace Corporation, ATR-72(9990)-1, March 29, 1972.

EQL, the Environmental Quality Laboratory, is an informally organized group of engineers, natural scientists, and social scientists who are dealing with broad, strategic problems of environmental control. Their "laboratory" is actually the world in which these problems must be solved. They interact with decision-makers in industry, government, and the ecology movement. Organized at the California Institute of Technology in 1970 in cooperation with the Jet Propulsion Laboratory, The RAND Corporation, and the Aerospace Corporation, EQL is supported by the National Science Foundation and private gifts.